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"SOME ASPECTS OF PLASTICITY"

Lynn S. Beedle*

A Condensation and Abstract of the Paper

"Ductility as a Basic for Steel Design"**

Introduction

One of the profound lessons coming out of attempts to make a conscious use of ductility in steel design is the extent to which it has always been inherent in design procedures. From the early days of his studies, Professor Baker has ~~himself~~ documented under 'shortcomings of elastic theory' many instances of this fact [1] and other examples are available [2]. The next step is fairly obvious: One should have a better understanding of the basic factors controlling the behavior of a structure whose design in the past has been satisfactory on the basis of experience, and this should be followed by the development of over-all design philosophy that recognizes the real strength of the structure.

It is the objective of this paper, through a review of Lehigh research, to show how it is possible to predict the real strength of structural elements, members, and frames utilizing the concept of a ductile material, to discuss the design implications resulting therefrom, and to note the extent to which the concept of ductility is utilized in design practice. ~~Rather than restrict the discussion to the topic that first comes to mind (in this connection (plastic design)), the scope will be broadened to give (a somewhat wider perspective).~~ In this discussion ductility is thought of as the ability of a material to absorb large plastic deformations without fracture, and for structural steel, would include the initial inelastic region, the so-called 'plastic' range, and the region of strain hardening.

Among the various steel research projects with which the Fritz Engineering Laboratory at Lehigh University has been associated, the

*Professor of Civil Engineering and Director, Fritz Engineering Laboratory, Lehigh University.

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following topics will be included: ~~(1) High-Strength Bolts,~~ (1) Centrally Loaded Columns, (2) Plate Girders, and (3) Plastic Design of Multi-story Frames. In each case the concept of ductile action comes into focus in a somewhat different way, and yet the basic objective of each project is the same: a better utilization of material in design.

Residual stresses and column strength†

There is scarcely a structural member that has been tested more extensively, that has been the subject of more controversy with respect to theory, and for which there is more variation in design approach throughout the world, than the centrally loaded column. The ~~arrival on the scene~~ Advent of welded columns and of steels with different yield points has merely intensified the confusion. In so far as design philosophy is concerned, the welded column represents a major unsolved problem, and the reason is that under certain circumstances some welded columns are ~~considerably~~ weaker than their rolled counterparts. Thus, if one wanted to have a completely rational design approach, it would be necessary to have a different formula for each class of columns, or it would be necessary to specify certain prior- or post-heat treatment procedures to columns, or else to require mechanical working after cooling. In the one case the design process becomes more involved. In the other the cost of fabrication is increased.

Residual stresses develop as a result of plastic deformation. In rolled shapes plastic deformation occurs during the process of cooling after rolling, and in welded shapes there is a similar plastic deformation after the welds are deposited. Since the part to cool last is in residual tension, then in both rolled and welded H-shaped columns the flange edges will be in residual compression.

Shown in Fig. 1 are typical residual stresses that have been measured in various rolled shapes. Although there are some variations, the important thing to notice is that at the flange edges in wide flange shapes the residual stresses are in compression. The low stresses in tubes (shown at the bottom of fig. 1) are the reason such members exhibit higher column strength.

Techniques have been developed to take into account the influence of these residual stresses on column strength, and they center about obtaining an effective stress-strain relationship [12]. ~~Plate 2~~ Fig. 2 shows by the dotted line the stress-strain curve for an ideal coupon, and by the solid line the average stress-strain curve obtained by testing a stub column, which is a full cross-section which is short enough to prevent column buckling but long enough to retain the residual stress. The curved line reflects the residual stress effect; and it has been established that the proportional limit is, in fact, the difference between the yield point stress and the compressive flange tip residual stress.

† In the following, there is no discussion about accidental eccentricity, a factor long known to have a significant influence on the strength of centrally loaded columns. In view of the pronounced variation in column strength due to the residual stress effect, major attention has been devoted to it. However, both accidental eccentricities and residual stresses must be considered in arriving at a design formulas~~, although at this stage there is no final agreement as to how this should be done~~.

At an intermediate stage in the inelastic portion of ~~plate 6~~ fig. 2 the cross-section will be partially yielded as shown in fig. 3. The fact that the yielded portions (shown black) can contribute nothing to the moment of inertia, is what gave rise to one of the basic concepts on the understanding of inelastic stability problems [13], namely, that the strength of a column with a partially yielded cross-section is a function of the moment of inertia of the portion that remains elastic, or,

$$P_{cr} = \frac{\pi^2 E I_e / I}{L^2}.$$

It has been shown that this, in turn, can be expressed in terms of the tangent modulus, E_t. Stated in a different way, column strength is going to be a function of the shape of the curve, because column strength depends upon the tangent modulus. Where there is a loss of modulus one must expect a reduction in column strength.

Figure 4 shows stress versus slenderness ratio curves for several conditions. The curve at the top is for flexure about the x–x axis (corresponding tests are shown by the squares). The lower curve represents weak axis flexure, with the open circles representing corresponding tests. The curve between the two is the CRC formula which is an approximation to the two theoretical curves [9]; it is the basis of much design practice in the U.S.A.

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INSERT A:

Figure 5 shows a picture of one of the test columns in the five million-pound testing machine at Lehigh. The Research Assistant is in charge of the test. All the research work at Lehigh is tied to our educational program, and every experimental or theoretical project is a part of an M.S. or a Ph.D. thesis program.

INSERT B:

The heaviest shape rolled in the U.S. (14WF730) is shown in Fig. ~~10~~ 6 at Lehigh on the main floor of our test laboratory. Mr. Brozzetti is shown with a colleague making residual stress determinations in this shape. It is so heavy that it cannot be tested even in our 5 million pound testing machine. After the preliminary measurements are made and after the residual stresses are measured, the other pieces (stub columns and full columns) will be tested at the National Bureau of Standards which is just now completing a testing machine with a capacity of 12 million pounds.

In the United States, as elsewhere, rapid developments are taking place in welded columns, and the ALCOA building in San Francisco (~~plate~~ fig. 7) is a ~~current~~ recent example ~~of considerable interest~~. This building features exposed diagonal bracing (a technique being followed in the 100-story John Hancock Building in Chicago). The ALCOA building is all welded, and many of the shapes are welded from three plates. It is to the stresses present in these plates and modified by welding that attention is now focused.

Figure 8 shows a comparison of stresses measured in two shapes. On the left are the stresses measured in a rolled shape (8 WF 31), and on the right, residual stresses measured in a welded shape (10 H 62). Compare 13·0 ksi compression in the former with 27·6 ksi compression in the latter. At the flange center of the welded shape the stress is about equal to the yield point in tension; it is this high stress that builds up the compressive stress at the flange tips to values that may reach as high as 2 to even 3 times the compressive stresses found in untreated rolled shapes. Of course, in thinking about column stability the important stresses are those in compression. Both theory and test confirmed what would have been expected on the basis of measured residual stresses: welded columns with higher compressive residual stresses showed lower strength [12].

Rather than penalizing all welded shapes, because one group of them exhibit a lower-than-average column strength, it might be of value to explore means of making these stresses work for us. One scheme is to change the distribution of stress through selected changes in fabrication technique. Figure 9 shows the result of one such approach in which flame-cut plates are substituted for untreated universal mill plates. On the left are the measured residual stresses in a welded shape built up from universal mill plates (rolled to required width). On the right are the stresses measured in a shape that was built up by welding from plates that had been flame-cut. (The heat in flame-cutting leaves residual tension at the edges of the flange.) Since the flange tips are no longer in compression, this should result in improved column performance.

Both the theoretical work and the test results confirmed these expectations. Referring to fig. 10 in which the loads are plotted against the slenderness ratio, the upper solid curve is the theoretical curve for one group of columns with flame-cut plates, and the lower curves are for welded columns with untreated universal mill plates [14]. (The dashed line is a weak-axis rolled shape prediction.) Corresponding test points have shown reasonable agreement, and we can conclude that the flame-cut H-shape is definitely stronger than the same shape welded from untreated universal mill plates.

Residual stresses are very much dependent upon geometrical factors, and one of the consequences is that larger shapes should be less sensitive to residual stress effects due to welding than smaller ones. If one maintains about the same size of weld, there is the same heat input and yet a much larger area to resist the tension force set up when the molten material contracts as it cools. So one should expect a smaller compressive edge stress.

The first results did not meet these expectations. The measurements that were made on one of these shapes with thick flanges are shown in fig. 11, the flanges in this instance being universal mill plates about 3 in thick and with full penetration automatic welds. The stresses are rather high—30 ksi on the flange tips. There is a considerable variation between the outside flange and the inside of the flange, due to the unsymmetrical location of the weld deposit.

One possible explanation of the fact that the stresses were higher than anticipated is because of the stresses that were present before welding. In universal mill plates the initial distribution would result in compressive flange tip stresses, and the welding would add to these stresses [15]. Figure 9. showed an example in which the fabricating of these heavy columns using flame-cut plates helped. Similarly, a controlled heating of the edges (without any cutting) might very well improve column performance.

The plastic action, resulting from cold straightening of columns, provides a further modification to the post-cooling residual stress pattern and there is every indication that column performance is improved thereby. Figure 12 shows the effect of rotarizing† on three different shapes. Since the usual compressive stresses are missing from the flange tips, and since any flange stresses are very low, then this is a distribution that favors a higher column strength [16]. In this connection the 'gagging' process is probably less beneficial than the rotarizing process because the latter results in continuous plastic deformation along the flange edges; gagging, instead, is quite localized.

Figure 13 shows the results of most of the column tests that are available to date [17]. It is shown in comparison with the AISC curve of 1963 [18]. After the foregoing brief discussion it is no surprise that there is a considerable variation in strength. There are some classes of columns that are definitely stronger than others: annealed, straightened, and tubular members. At the bottom of the group of test points are the welded H columns with normalized plates. This leads to the question that perhaps higher stresses should be permitted for some columns than for others.

The welded wide flange of normalized universal mill plates exhibits the lowest strength and perhaps a lower limit curve should be established for this category. Next might come welded columns with low heat-input, welded columns with flame-cut edges, and then rolled wide flange columns. Perhaps these should be grouped into a second column curve. Column strength is further improved in riveted and bolted columns, and (relatively) in columns of higher strength steel, in cold straightened members and in rolled boxes and tubes. At the top of the list, the strongest would be the annealed columns in which the residual stresses are eliminated. Perhaps these should be grouped into a third column curve. This is all supposition, of course, a supposition that assumes that there is nothing that can be done to modify these stresses.

† 'Rotarizing' is a method of straightening a shape by passing it through a series of offset rolls which deform it alternately by reversed plastic deformation.

However, there is another approach, and that is to make plasticity work for us. Through more refined control of cooling after rolling, through controlled upsetting due to preheating or post heating, or through plastic deformation introduced by cold working in conjunction with the straightening process, it should be possible to introduce a residual pattern for which the corresponding column strength might well attain a higher value, but which would still be satisfactory for design.

The choices then are these: (1) establish the column curve as the lower limit and ignore the predictably higher strength of other classes of columns, (2) institute controls on fabrication to assure a predetermined column strength, (3) establish several column curves which will be applicable for certain groups of columns. Discussions now in progress and the completion of current research will determine which direction United States design practice will take. In any case it will be a 'limit' design. Column formulas have always been based on ultimate strength, and not infrequently in strong dependence on test results, a fact powerfully underscored by the most recent efforts of the European Construction Steelwork Association program ~~described below~~[10].

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Participation in the program on "European Column Tests" on a cooperative basis is of a particular interest because it will enable us to make a detailed comparison of the different philosophies of column design, of column analysis, and of column testing that are in vogue today. The exchange of information has already begun, tests on some of the U.S. shapes have been completed, and work will start on the European shapes as soon as these are received.

Plate girders

Research on plate girders started at Lehigh University in 1957. Th first of these studies was on vertically stiffened plate girders of whicl the one shown in ~~plate 5~~ Fig 14 is a typical test, the members being loaded i pure bending [19]. The height of the girders in this figure is 50 in and th web thickness is $\frac{3}{16}$ in. The subject of the first study was to confirm tha buckling of a girder web does not mean failure. As a consequence th web thicknesses used in the tests were quite thin.

If web buckling does not cause failure, then what does fail? It is the compression flange, and it can fail in a number of ways: by torsion, by lateral buckling, and by vertical buckling. With a very stiff flange it is possible to develop considerable 'plastic hinge' action as shown by the curve of fig. 15.

Although no attempts have been made as yet to utilize the 'plastic hinge' concept in plate girder design, current United States practice very definitely utilizes the concept of redistribution. When the web of a plate girder 'buckles' in the sense that the compression zone moves sidewise, it cannot carry additional stresses. Instead these stresses are redistributed to the compression flange. The design application of this research in current use (AISC) is to specify stress reduction factors to apply to the flange, such that it will be able to support the stresses redistributed to it by the deflected web plate.

In addition to the flexural studies, the behavior of girders in shear has also been examined. Figure 16 shows the shear strength as a function of the panel length-to-depth ratio, for a given depth-to-thickness ratio. The lower of the two curves is for buckling and the upper one is for development of superimposed tension field action; the distance between the two represents the post buckling strength that is available in a plate girder under shear loading. The bars represent the results of tests confirming the post buckling strength that comes about as a result of tension field action.

Figure 17 shows a typical load-deflection curve for a plate girder in shear. Shown in the figure is the buckling stress V_{cr} (note the considerable margin of safety) and the uniform plastic shear failure load, V_p, considerably above what a specimen with such a thin web ($b/t = 260$) could be expected to support. Of interest is the basis for predicting V_u on which rests, with suitable factor of safety, the currently recommended design stresses.

A method for determining P_u has been developed making use of the finding that total shear force, V_u, can be accurately approximated as a combination of 'beam action' shear and 'tension field' action, or $V_u = V_\tau + V_\sigma$ [20]. The upper limit of the former is taken as the critical buckling load, V_{cr} (which is assumed to remain constant on further loading). The upper limit of the tension field force is reached when a superposition of stresses, resulting from the two actions, reaches the yield condition, and unrestricted flow would then occur over the tension field strip. ~~Plate 6~~ Fig 18 shows the tension field that develops under extremes of deformation past ultimate in a yielded shear panel. The strip geometry derived in the theory is superimposed on the photograph.

Thus there are really two 'limit' conditions involved in the failure analysis of a plate girder under shear loading. The first is a buckling condition, and the second, unrestricted plastic flow. Of course, the load-deflection behavior of a real girder does not reveal any sudden discontinuity at V_{cr}. Lateral web deflection and tension field action commence at very low load levels, and, in fact, it has not been possible from any measurement to detect the onset of buckling. In spite of this, the limit approach works very well as shown by fig. 16. Very good agreement between theory and tests of five large plate girders was obtained for aspect ratios from 0·5 to 1·5 and a depth-to-thickness ratio of 260[20]. A recent revision to the theory gives even better correlation.

Research on longitudinally stiffened plate girders provides yet another example of the ability of one portion of a steel structure to deform without change in load while other elements develop their strength, thus increasing the total capacity. It is evident in plate 19 that the horizontal stiffener has forced a tension field to develop in each of the subpanels. The full capacity of one of these subpanels was reached first. This panel deformed in a ductile manner while the second panel gradually attained its full strength. In some of these tests the increase in load capacity due to longitudinal stiffeners was as much as 25%[21].

Plastic design of multi-story frames

The first authorization for the use of plastic design in the United States was based on the Proceedings of a conference held in 1956 at Lehigh under the auspices of the American Institute of Steel Construction [22]. In 1958, a Supplement to the AISC Specification was issued [23], coinciding with the visit of Sir John Baker to present a summary of the most recent British work [24]. In 1961 that which started out as a supplement became part of the main body of the Specification. The method had come of age, so that one could speak in terms of two methods of design: 'Allowable Stress Design', which was Part 1 of the Specification, and 'Plastic Design' which was Part 2. It is of particular note that many of the provisions of 'Part 1' were influenced by the work that had been done on the maximum load-carrying capacity of members and frames.

In so far as plastic design of multi-story frames is concerned, the 1961 and 1963 editions of the AISC Specification contained limited allowance for use of the method. It allowed the designer to proportion continuous beams in a multi-story structure by the plastic method, the column being required to 'remain elastic' [18].

The principal effort at Lehigh since 1958 has been on two major extensions of plastic design—the application of the method to structures utilizing the higher strength steels and to extending its use in the design of multi-story frames. The latter research has been divided into two phases: 'Braced Frames' where the resistance to lateral load (and to frame buckling) is provided by diagonal bracing (which could be X-bracing, K-bracing, or shear walls), and 'unbraced frames' in which the frame depends on the bending resistance of the frame members themselves to resist lateral load.

Work progressed sufficiently to the point that in 1965 a ten-day summer conference was presented at Lehigh [25]. There was representation from a considerable number of countries; and in fact, numerous delegates from abroad participated as speakers in a special lecture series organized as part of the conference [26]. A set of lecture notes developed for the conference contained the theoretical basis and the techniques developed for the plastic design of multi-story frames [27]. Figure 20 is typical of the structures that were designed to illustrate the method, a method that utilizes the maximum strength of both girders and columns.

In addition to the lectures and group discussion, the conference featured an extensive series of tests to verify the theoretical material [25]. Figure 21 shows one of these test frames. It is a frame braced against lateral motion by diagonal bars from corner to corner to provide the resistance to horizontal load. The story height is 10 ft and the bay spacing is 15 ft. The beams were 12 B 16·5 shapes and column sections were 6 WF 20 and 6 WF 25. Both vertical and horizontal loads were applied.

The results of the test are shown in fig. 22. The load on the beam is plotted against the centerline beam deflection. The dotted line is the theoretical prediction, and the line through the plotted point shows the test results. Correlation with the theory was very satisfactory [16].

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the ~~[illegible]~~ revision to the AISC Specifications, in addition to being broadened in scope to permit plastic design in higher strength steels, ~~[illegible]~~ incorporates provisions permitting design of braced multi-story frames. As a matter of fact, more than one such structure has already been designed, having been based on the previously mentioned *Lecture Notes* [27]. A manual describing the method which will be suitable for design office use ~~[illegible]~~ is now available ~~[illegible]~~ [28], and revisions are now being completed ~~[illegible]~~ to the "Commentary" to bring it up to date with the latest findings [29].

Some of the most critical problems in multi-story frame design have to do with the combined influence of axial force and deformations. This is especially true in the case of unbraced frames which undergo significant lateral motion when lateral loads are applied (fig. 23). The axial load times the deformation introduces additional moments to the columns, and this in turn results in a lowering of strength, leading to instability effects. This is the first problem, that of taking into account the so-called '$P-\Delta$' effect.

The second problem arising from the axial load effect is frame buckling and is very similar to the sudden buckling that is characteristic of column behavior. It is simply modified by the fact that the columns are restrained.

Figure 24 shows the results of a test of a single story frame with extra load applied to the column top to increase the $P-\Delta$ effect. The columns in the frame were 9 ft in height and the span was 15 ft. The beam was of high strength steel, and structural grade carbon steel was used for the columns. There was excellent agreement between the experiment and the theory which took into account this $P-\Delta$ effect [31, 27]. Ref 32.

The second problem, frame buckling, has been studied extensively, and Some of the most recent Lehigh work is contained in ~~[32]~~. Fig 25, ~~Plate~~ shows one of the frame stability specimens after test. Of course, the axial force is highest in the lowest story (the column section is uniform, and the column base is pinned). So the sway took place in the bottom story. The results of the test are shown in fig. 26. On the left, compare the maximum load reached with the value $P_u = 26{\cdot}6$ kips, the simple plastic theory load. On the left, the load is plotted against the vertical deflection of the beam; to the right, the deflection is that due to sway. Compare the same observed load (23·5 kips) with 23·6 kips, which is the critical load predicted by the theory.

Some of the largest tests conducted on this project at Fritz Engineering Laboratory in recent years have been on unbraced frames in which the resistance to lateral motion is provided by the bending strength of the members. Fig 27 ~~Plate~~ shows one of these frames (in white), a three-story, two-span structure using 6 WF 25 columns, 12 B 16·5 floor beams, and 10 B 15 roof beams. At the end of application of the horizontal load, the deflection was nearly 10 in. Figure 28 shows the results of the test. The result is slightly above the theoretical curve that takes into account the $P-\Delta$ effect and is, of course, below the first-order theory curve. (The solid dots in fig. 28 are the points at which plastic hinges theoretically should develop.)

Although methods of analysis of unbraced frames are well developed, there still remains some work to be done to obtain experimental confirmation of special cases and in particular to make the design approach adaptable for practical applications [33,34].

One of the complicating factors is that some of the design checks require a consideration of the continuity condition. In a way this is distressing, because the substitution of the mechanism condition for the continuity condition contributed to the essential simplicity of the plastic method. However, if one is to obtain a design method that is based on maximum useful strength, then the continuity condition must be considered for subassemblages. As shown by fig. 29 continuity of joint rotation makes it possible to consider the case where the subassemblage as a whole is stable, even though the column as an isolated unit has commenced to unload. In this instance, the adjoining members provide the needed support [35,16]. In actual design, charts will be used to aid in the solution [36].

What about the weight of a plastically designed frame compared with a frame designed on an allowable stress basis? A comparison for a ten-story unbraced frame is given in fig. 30 The weight of the frame designed by the plastic method is shown by the hatched bars, the allowable stress design by the open ones. For the girders, the saving in weight is about 20%, the difference for the columns is 7%, and the over-all total weight saving is about 12%. A similar comparison for a braced frame showed a weight saving for the entire structure of $6\frac{1}{2}$%.

Summary

Each element or system that was examined in this article was similar in behavior to that which is sketched in fig. 31, even though the situation is for a gabled frame. There is an initial elastic region, there is a region of elastic-plastic or contained inelastic deformation, and finally there is a region of plastic flow in which the structure attains the maximum strength. ~~Except for the slip zone in high-strength bolted joints~~ The transition from one region to the other is gradual.

Here

In spite of the significant differences in structural elements that were considered in this discussion, each in its own way has demonstrated that remarkable quality of ductility in steel structures which makes it possible for a portion to deform, permitting other elements to develop their strength as well, thus contributing to an over-all increase in the strength of the structure as a whole. The following summarizes the similarities and the differences:

Indent this part

(1) Columns yield locally, ~~at their edges~~, redistributing stress to the remaining elastic portions until eventually (if the column is short enough) the entire cross-section reaches the yield plateau. Most practical columns do not fail in the elastic region, but they are sufficiently long so that they are sensitive to fabrication effects (rolling, welding, heat-treating, straightening) and instability occurs before the redistribution process is complete.

(2) Plate girder webs buckle 'locally' and subsequently redistribute stress to other portions. Those in shear also develop tension fields (sometimes successively) and finally flow plastically at an ultimate load that can be calculated based on the sum of the buckling and tension field contributions at the plastic limit.

(3) Multi-story frames form yield hinges and redistribute forces to other parts of the frame in a manner more consistent with the ordinary plastic theory concepts. In some instances, in the final design check, it is necessary to return to a consideration of the continuity condition in order to determine that local failure does not prevent the structure from attaining the desired ultimate load.

What then is to be used as a basis for selecting the design load in a structural member or frame? In general, it will be based on the behavior of the structure and by the limit of usefulness that reflects this behavior. What are these possible limits of usefulness? For one, there is a deflection limit. Secondly, there could be fatigue or fracture. Thirdly, there could be instability as in the case of frame instability or column buckling. And if none of these occur, there would be, finally, the attainment of maximum strength (fig. 31).

Consideration of these various limits of usefulness has led in the past to two major design philosophies, allowable stress design and plastic design, and to another major design criterion, a design for stiffness (a deflection limit).

Eventually, it would appear likely that there should be only two major design criteria—a design for strength, which will be maximum load design, and a design for stiffness. There should be less and less reference to 'elastic design', since the elastic limit has no particular significance in the load-deformation behavior.

In the meanwhile what is really needed is a better definition of 'failure', and this can be achieved by (1) acquiring a better understanding of the behavior of structures and their elements and (2) by an improved formulation of design requirements. How much deflection can be permitted in a structure? For how many cycles of stress fluctuation must a structure be designed? What overload safety factor is required for the various functional, loading, and geometric conditions? These are topics that need additional study. Further knowledge of structural performance needs to be developed, especially with respect to three-dimensional frameworks.

We are fortunate to live in an age of rapid technological change. The results of research are incorporated into design fairly rapidly as a consequence of continuous cooperation between industry, the government, research councils, universities, and even between different countries. As a result engineers are provided with the best possible information for the efficient and safe design of structures.

Acknowledgments

The work at Lehigh University summarized in this paper is essentially the work of the writer's past and present colleagues at the Fritz Engineering Laboratory. The opportunity to summarize their work is gratefully acknowledged. The support of the steel industry and of various branches of the government in the conduct of the research is similarly acknowledged. Portions are based on a survey prepared for the Japanese Society of Steel Construction, with special acknowledgment due to Yuzuru Fujita, Fumio Nishino, and Koichiro Okuto. In connection with this particular manuscript the assistance of William A. Cranston, J. Hartley Daniels, Richard Sopko, Mrs John Fielding, and Miss Flo Ann Saeger is noted with appreciation.

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On a more personal note, the writer has known Mr. Sfintesco for more than three years now, and it was a particular pleasure to be able to visit him in his own country on which occasion this lecture was given. The hospitality of Mr. Sfintesco, Prof. Lorin and Mr. Wahl on the occasion of that visit is greatly appreciated; Paris lived up to its reputation as one of the truly great cities of the world. The assistance of Mr. Brozzetti, Research Assistant at Fritz Laboratory (on leave from CTICM) is gratefully acknowledged.

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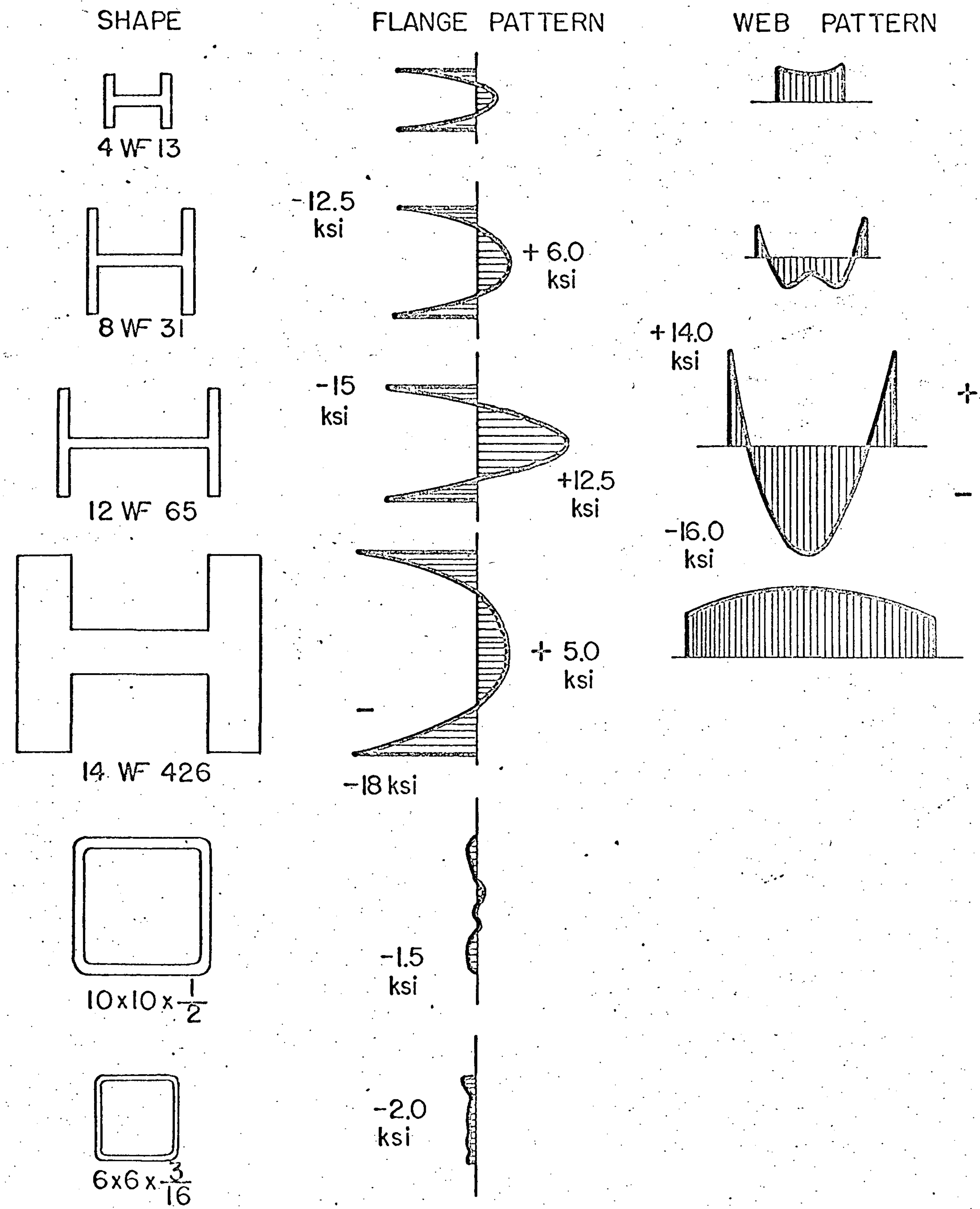
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Fig. 1

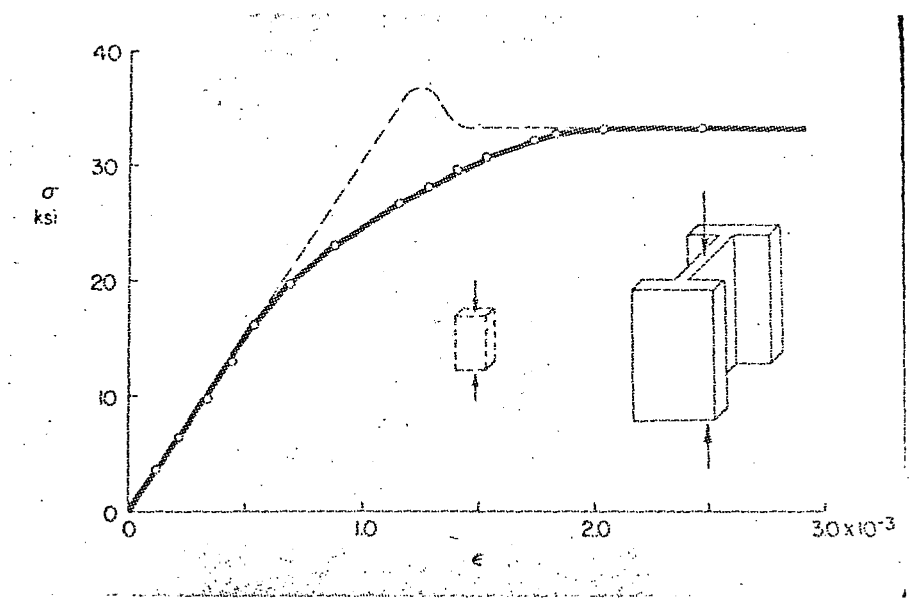

Fig 2

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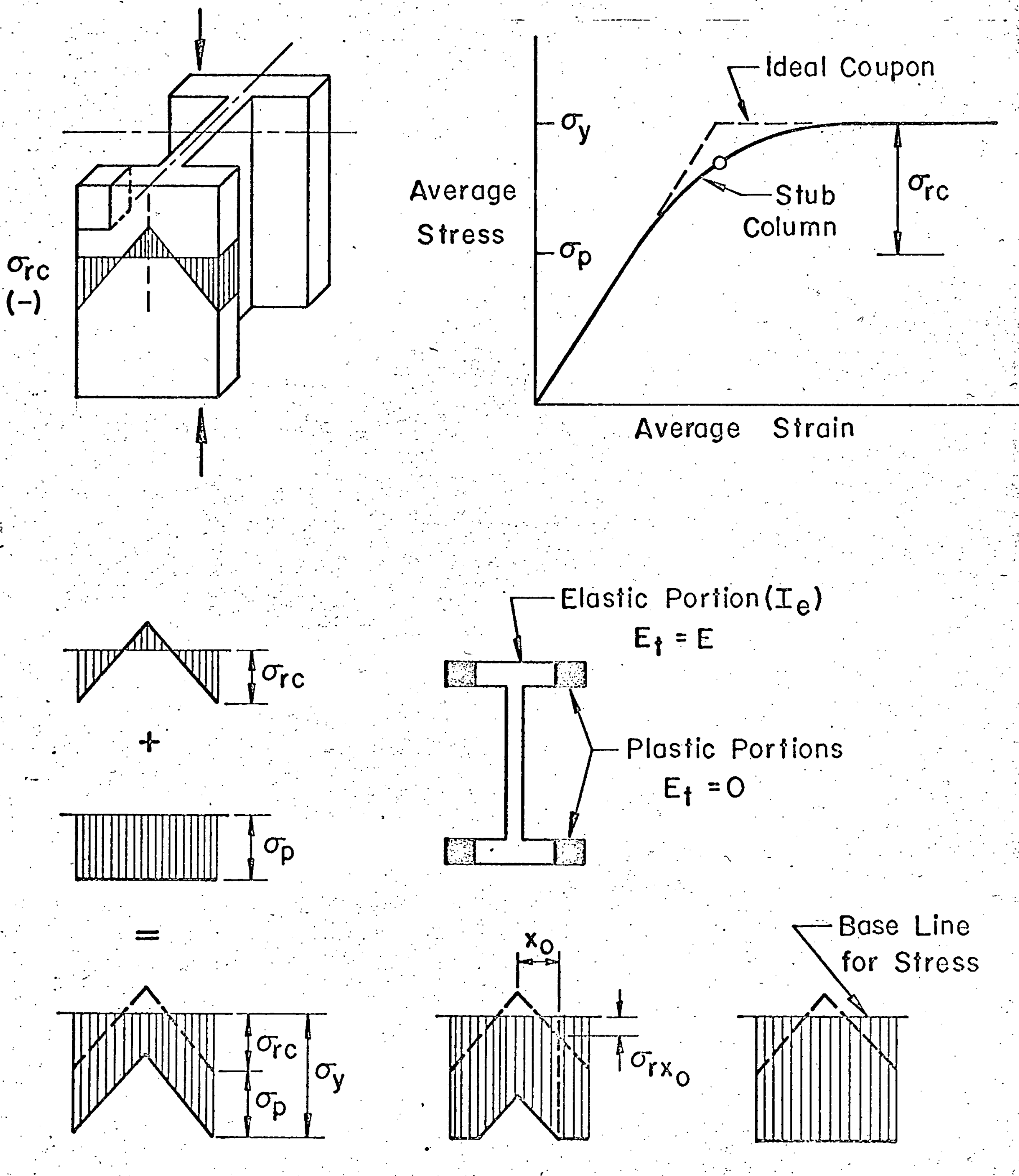

fig. 5

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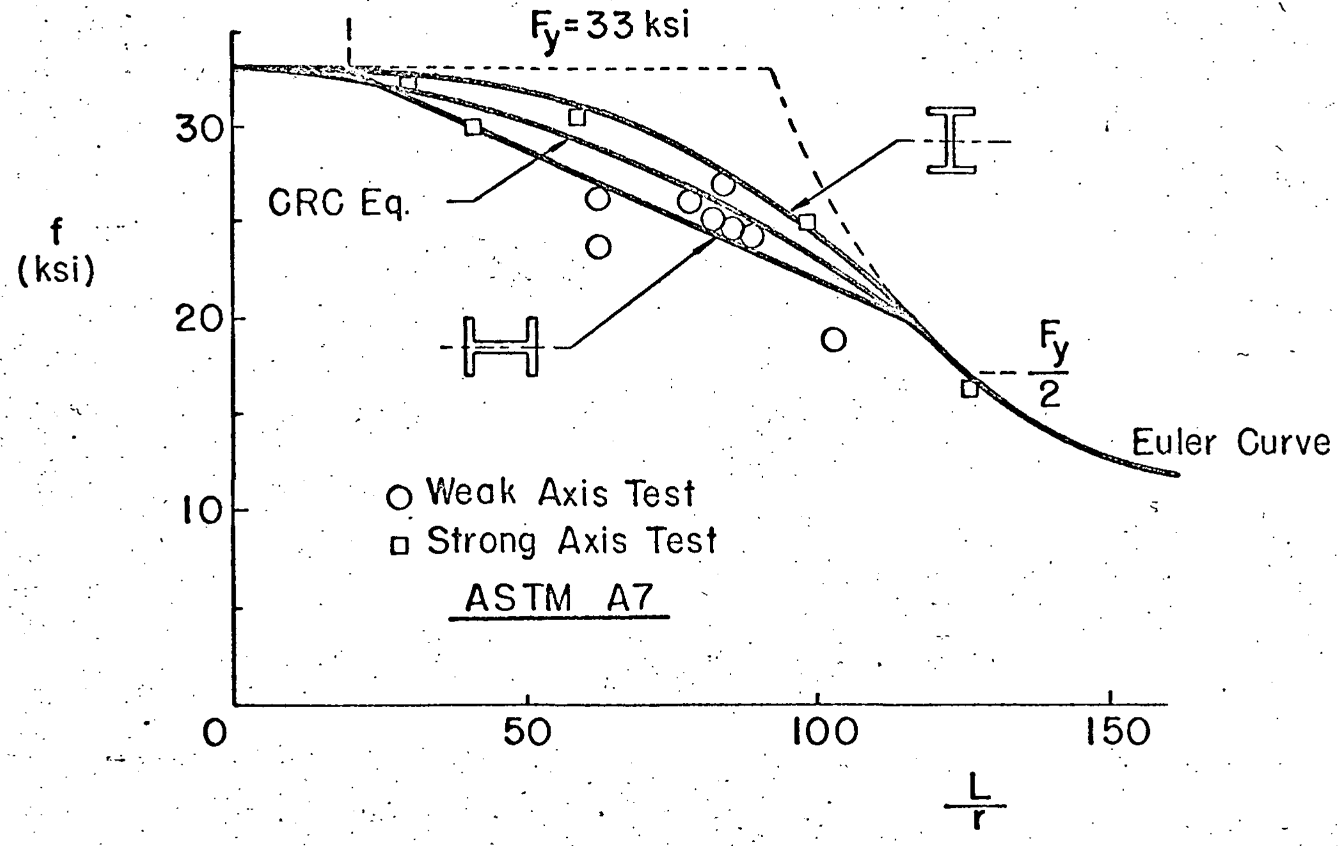
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$\frac{L}{r}$

Fig 4

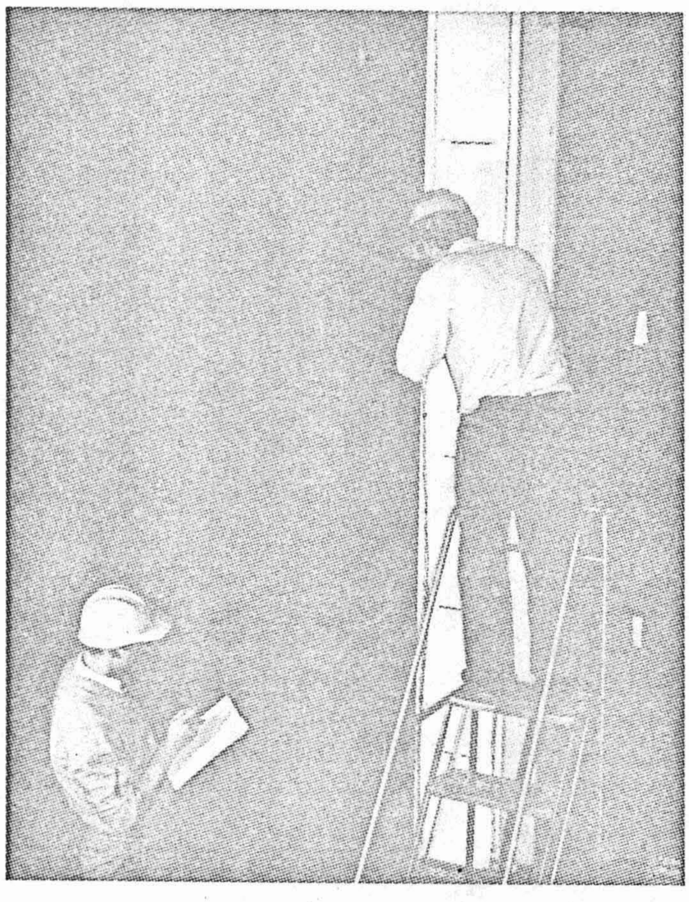


Fig 5

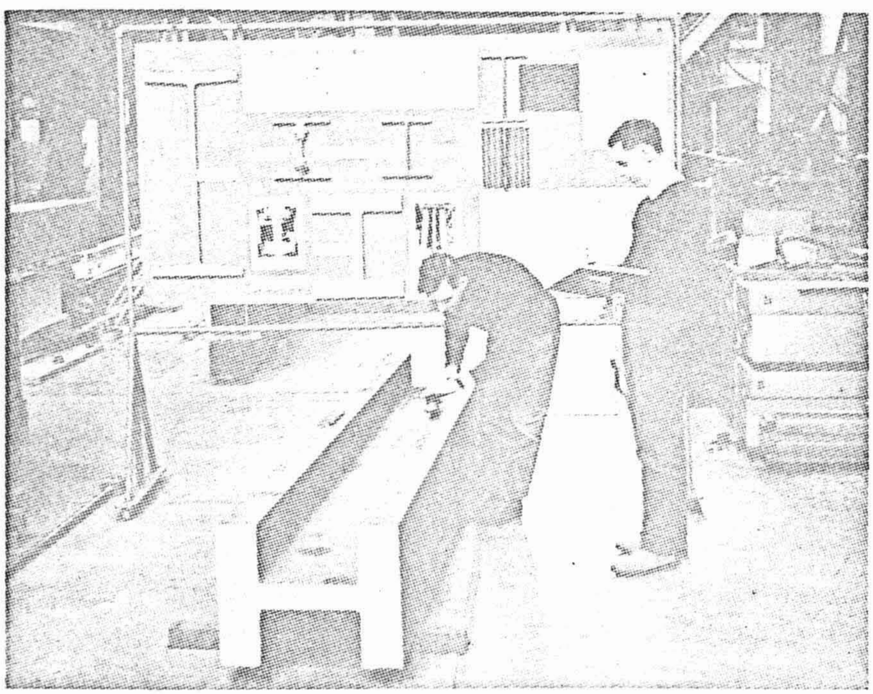


Fig -6

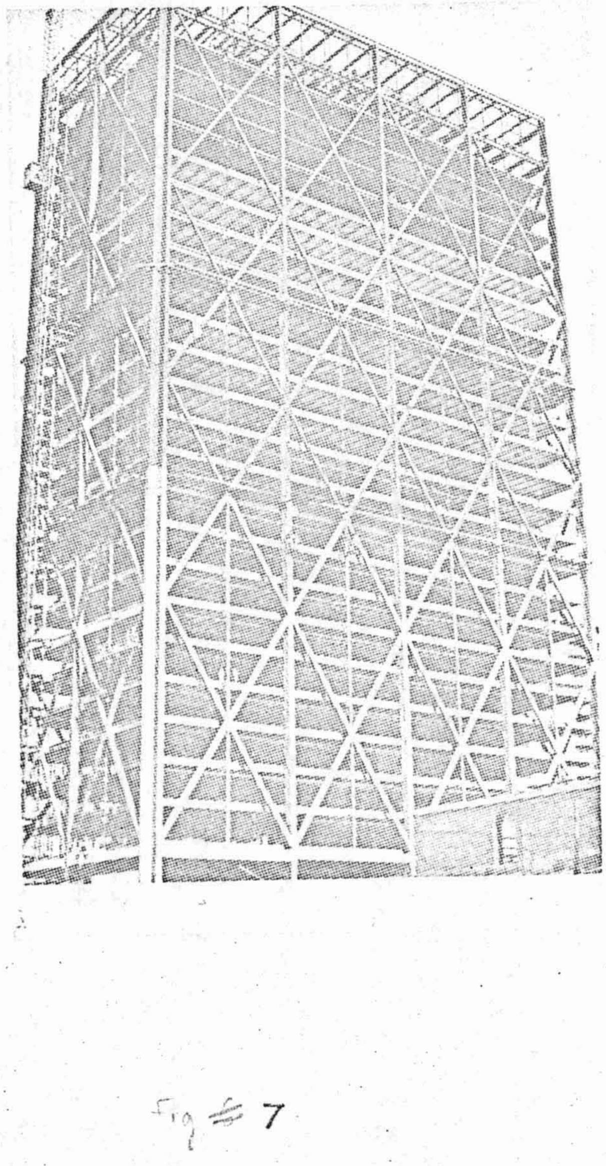


Fig # 7

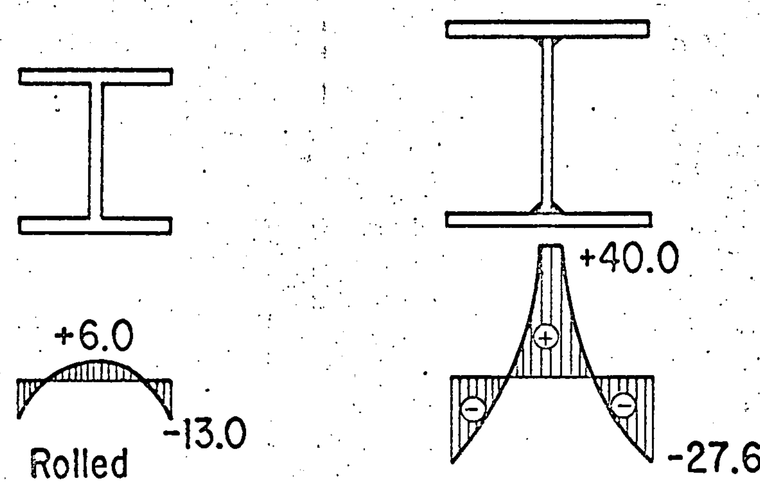
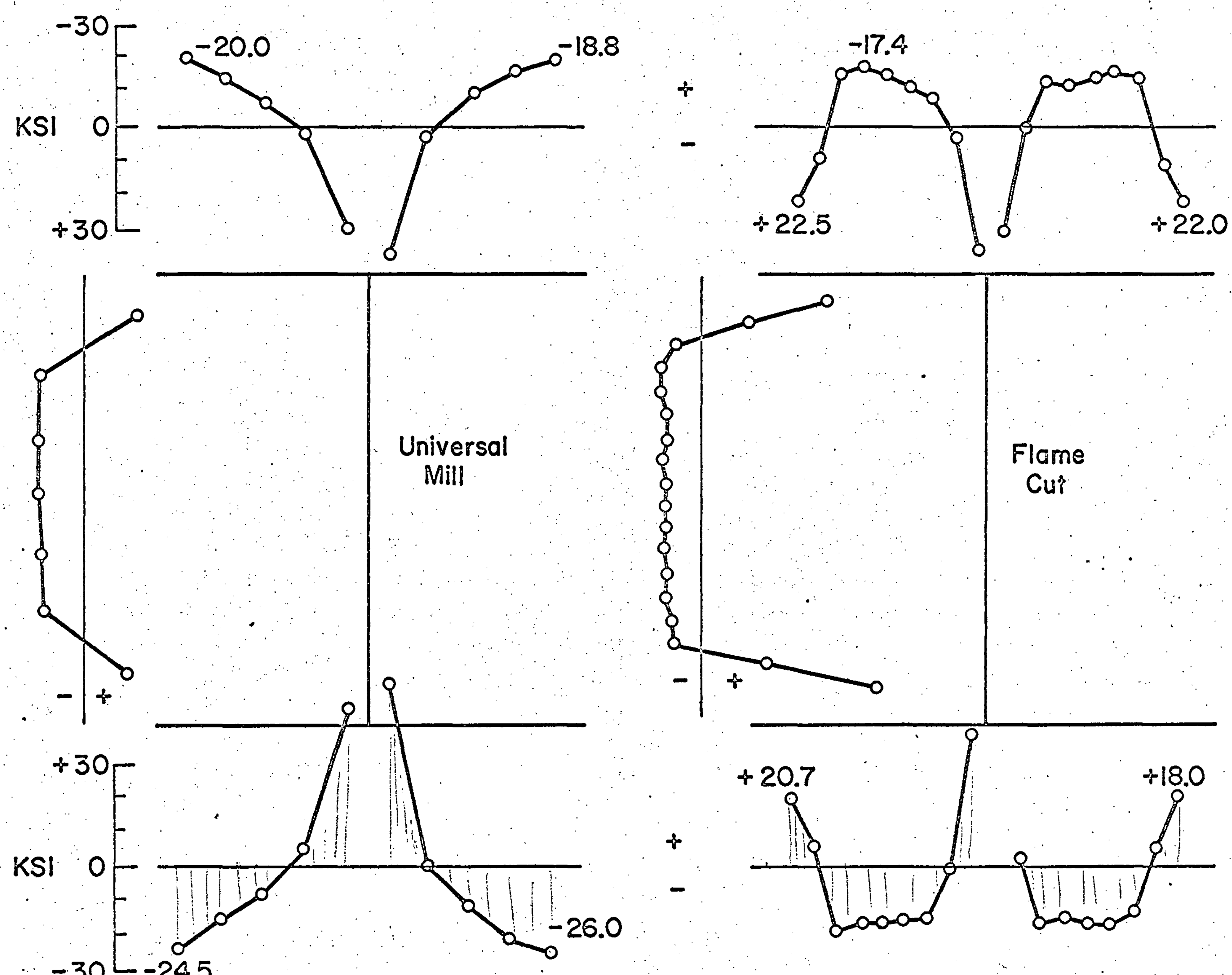
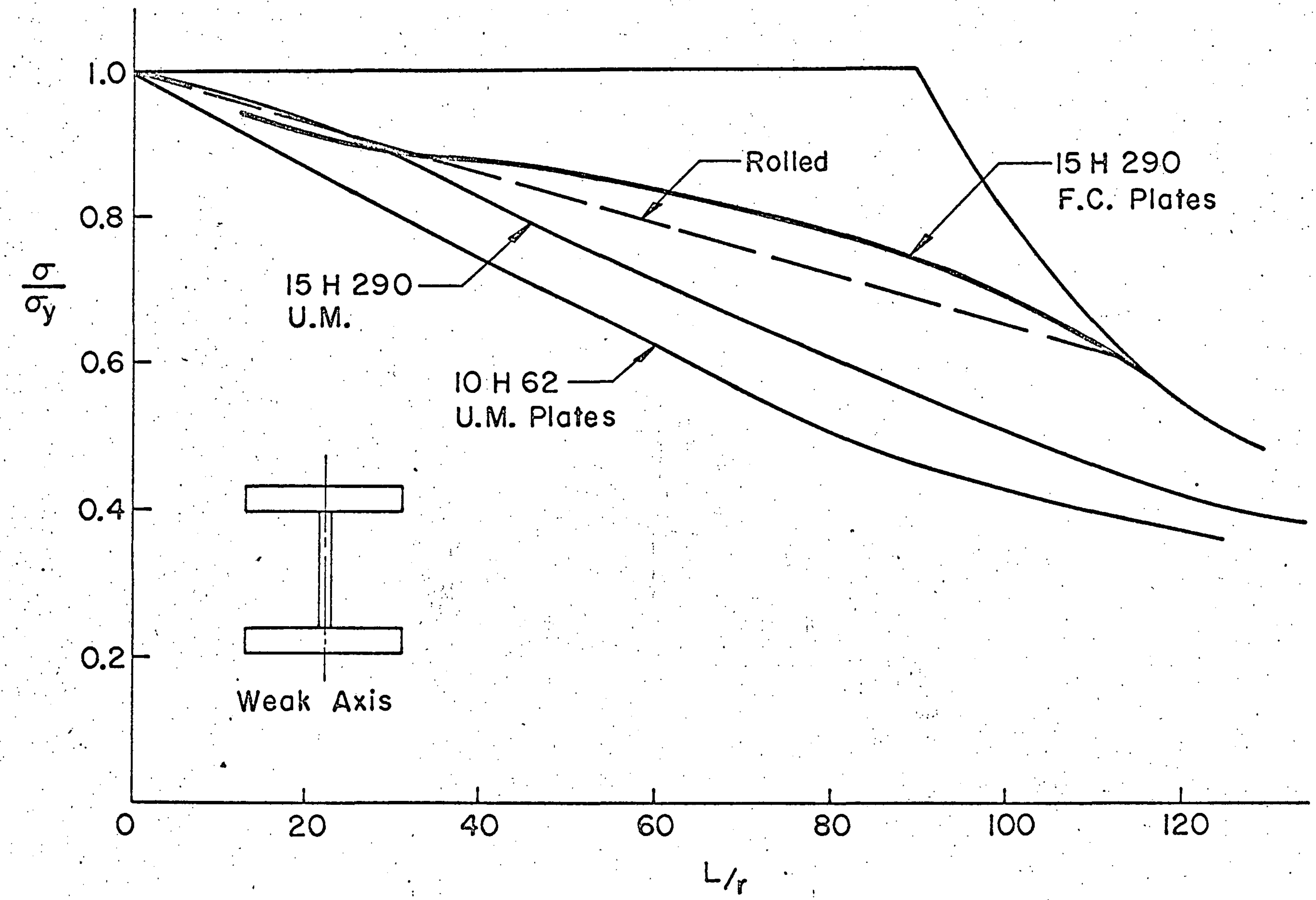


Fig 8

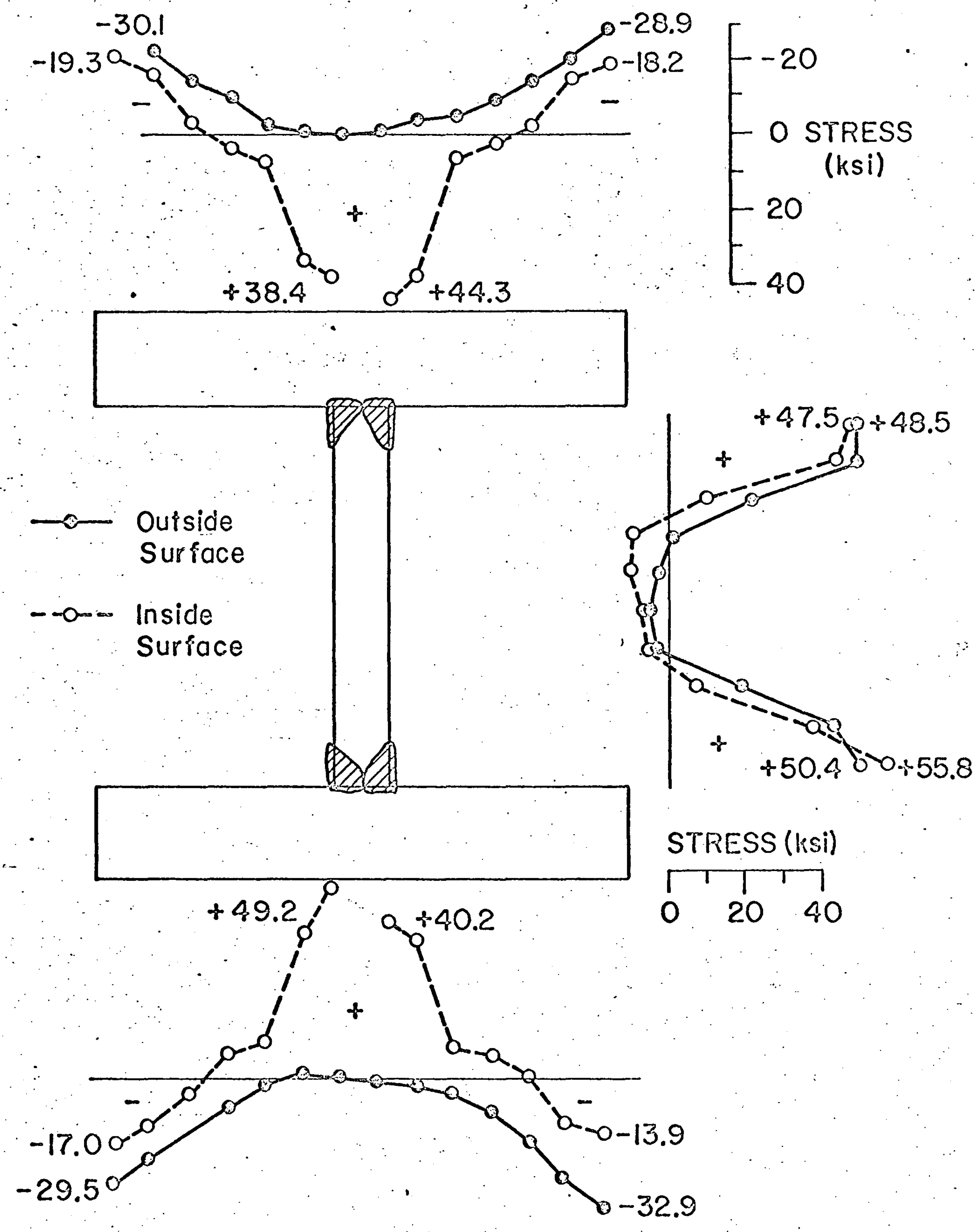

-30
-20.0
-18.8
KSI
0
+30
-17.4
+
-
+22.5
+22.0
Universal
Mill
Flame
Cut
-
+
-
+
+30
+20.7
+18.0
+
-
KSI
0
-26.0
-30
-24.5

10 H 62

Fig 9


1.0
0.8
0.6
0.4
0.2
$\frac{\sigma}{\sigma_y}$
Rolled
15 H 290
F.C. Plates
15 H 290
U.M.
10 H 62
U.M. Plates
Weak Axis
0
20
40
60
80
100
120
L/r

249.12-1



15 H 290
A36 STEEL ; GROOVE WELD
U.M. PLATES

Fig 11

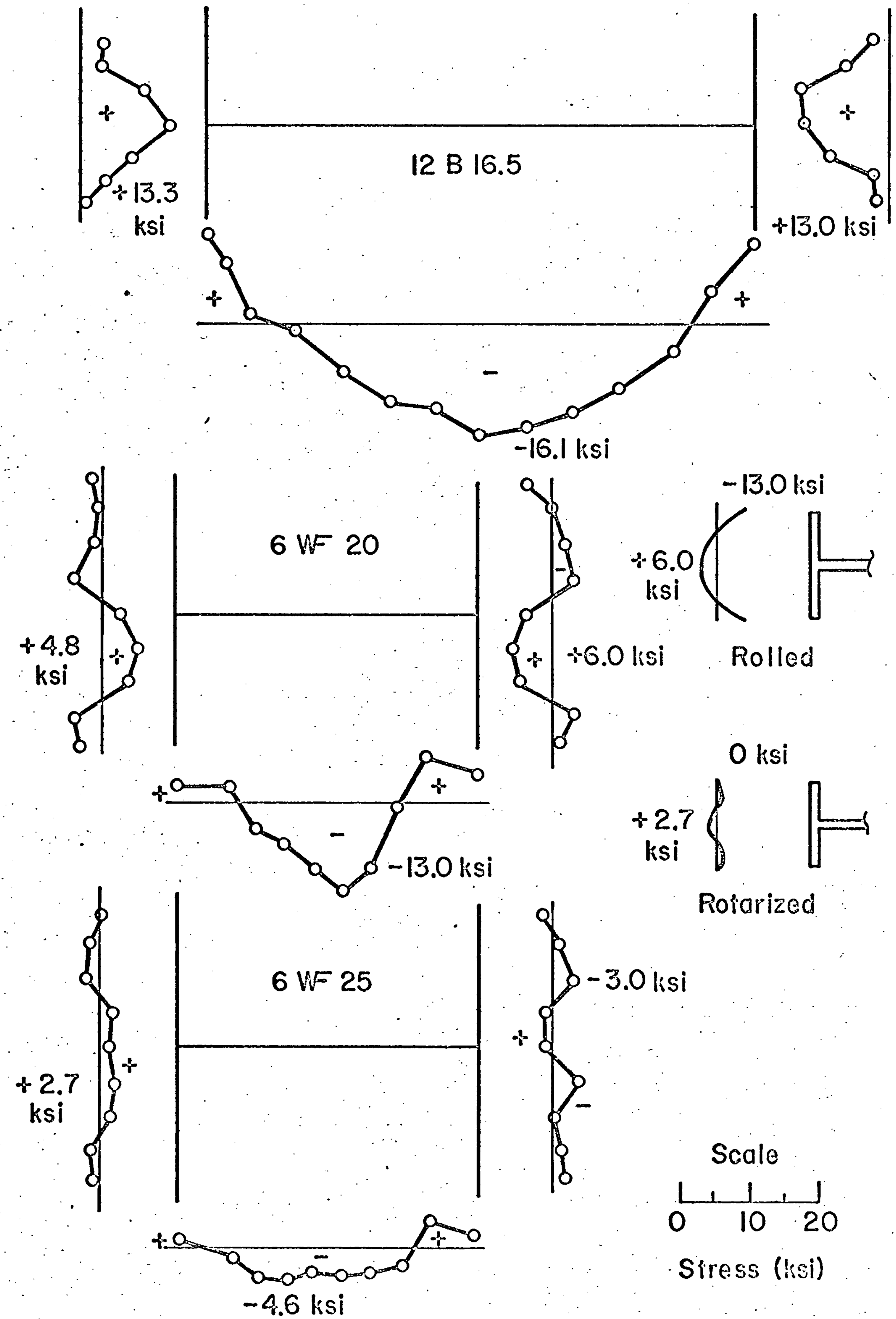

12 B 16.5
+13.3 ksi
+13.0 ksi
-16.1 ksi
6 WF 20
+4.8 ksi
+6.0 ksi
-13.0 ksi
-13.0 ksi
+6.0 ksi
Rolled
0 ksi
+2.7 ksi
Rotarized
6 WF 25
-3.0 ksi
+2.7 ksi
-4.6 ksi
Scale
0 10 20
Stress (ksi)

Fig 12

249.105

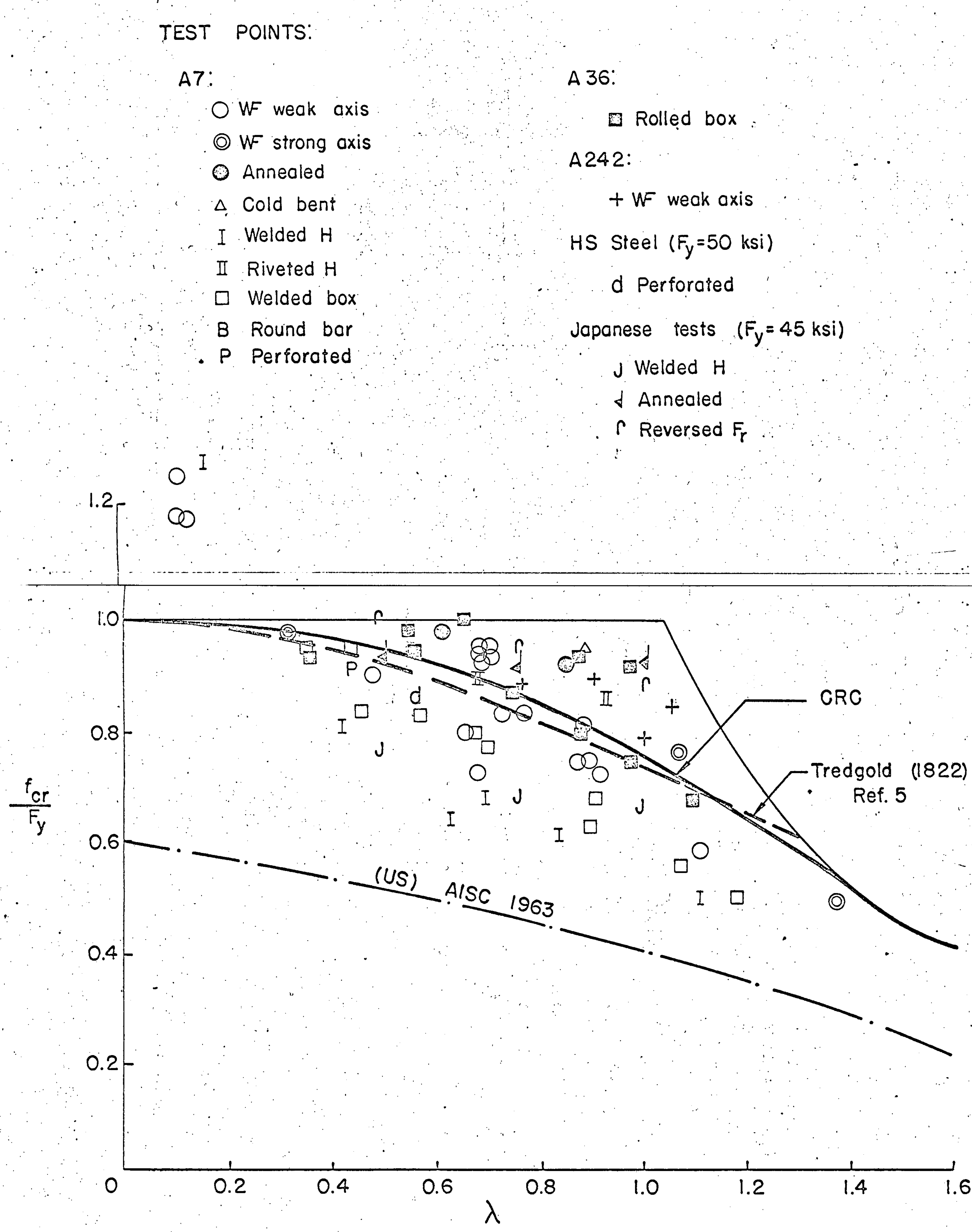


Fig 13

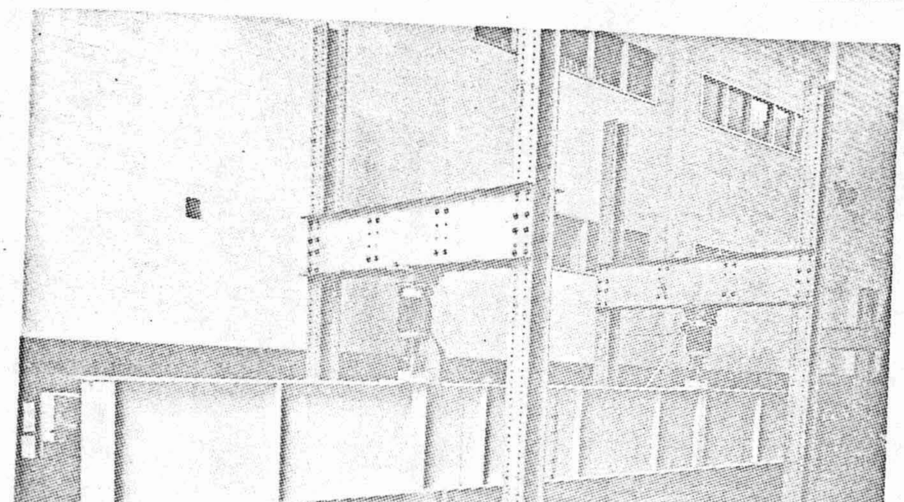
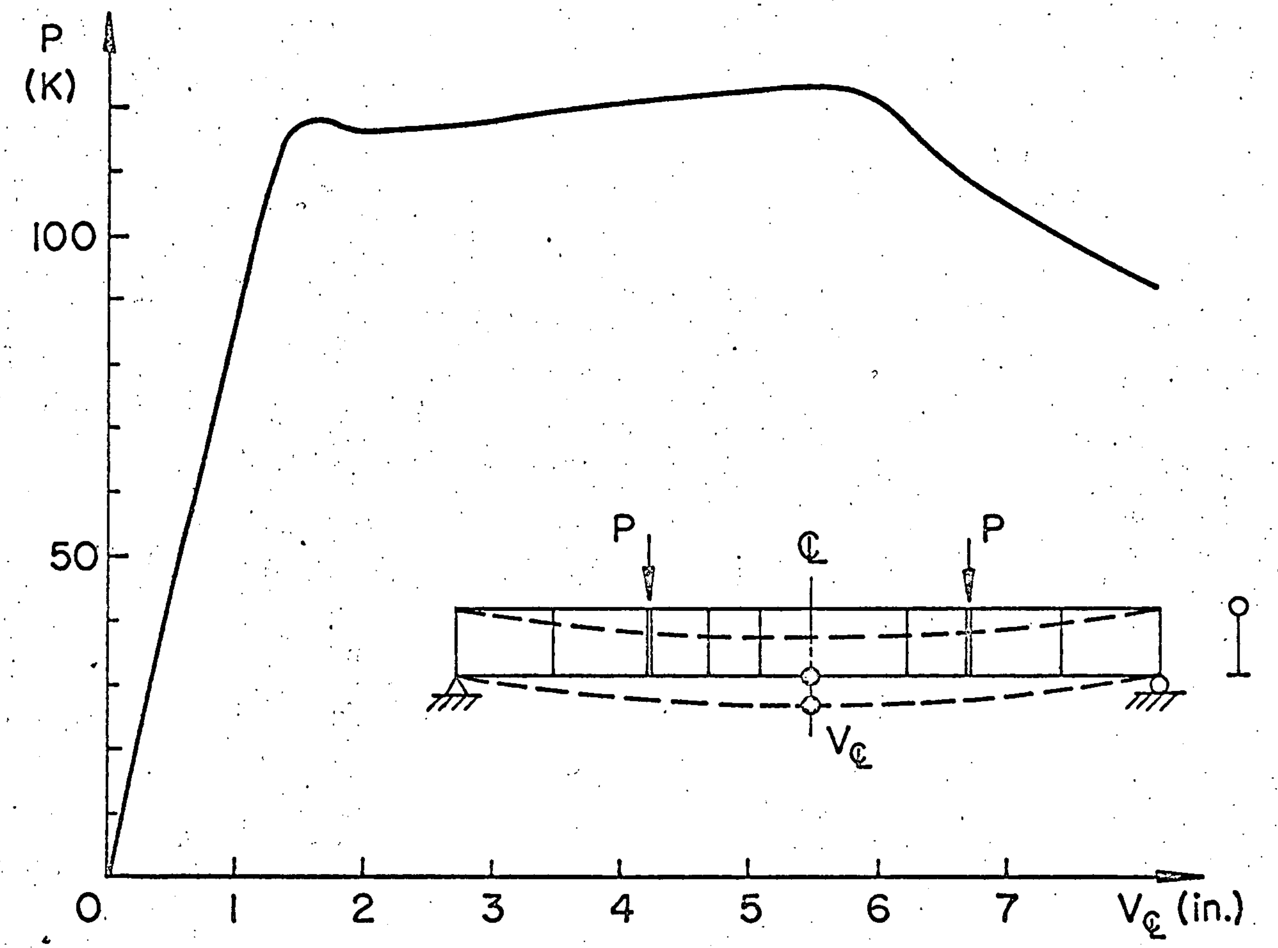
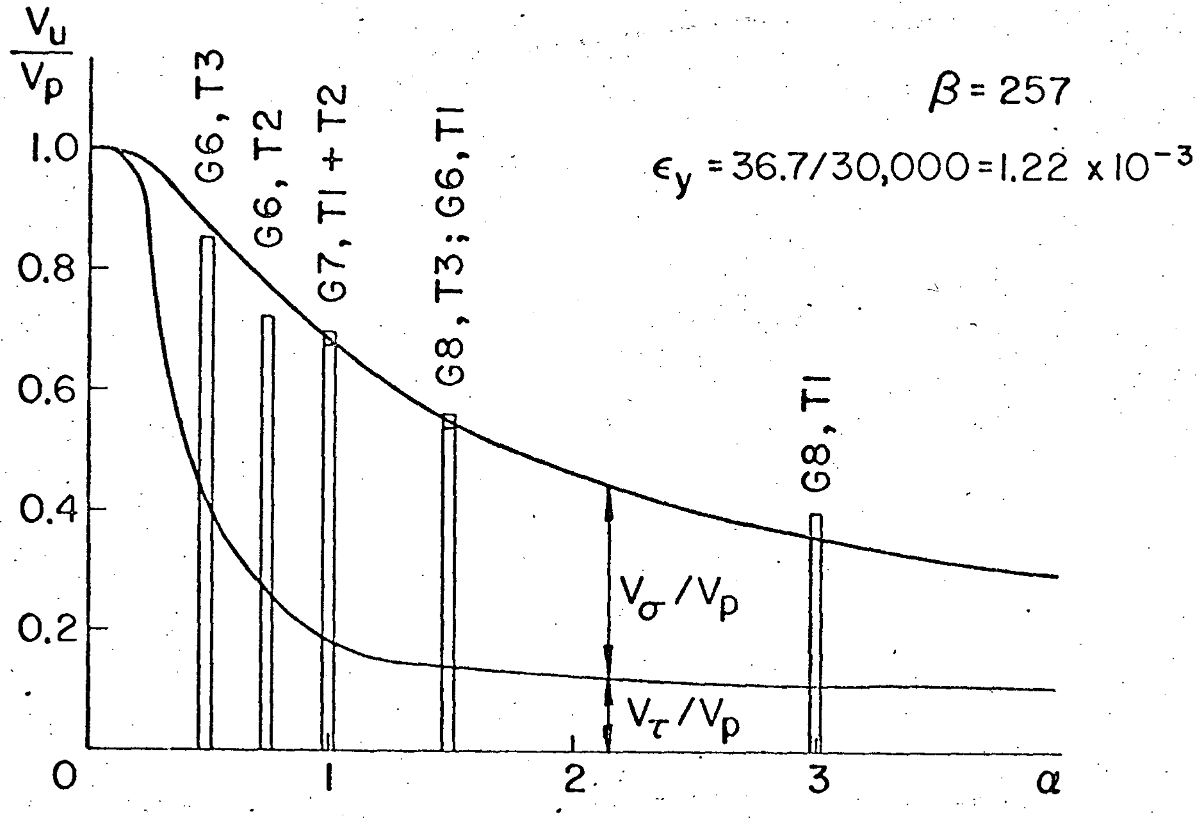


Fig. 14



— Fig 15 —



$\frac{V_u}{V_p}$
1.0
0.8
0.6
0.4
0.2
0
1
2
3
α
G6, T3
G6, T2
G7, T1 + T2
G8, T3; G6, T1
G8, T1
β = 257
$\epsilon_y = 36.7/30{,}000 = 1.22 \times 10^{-3}$
V_σ / V_p
V_τ / V_p

Fig 16

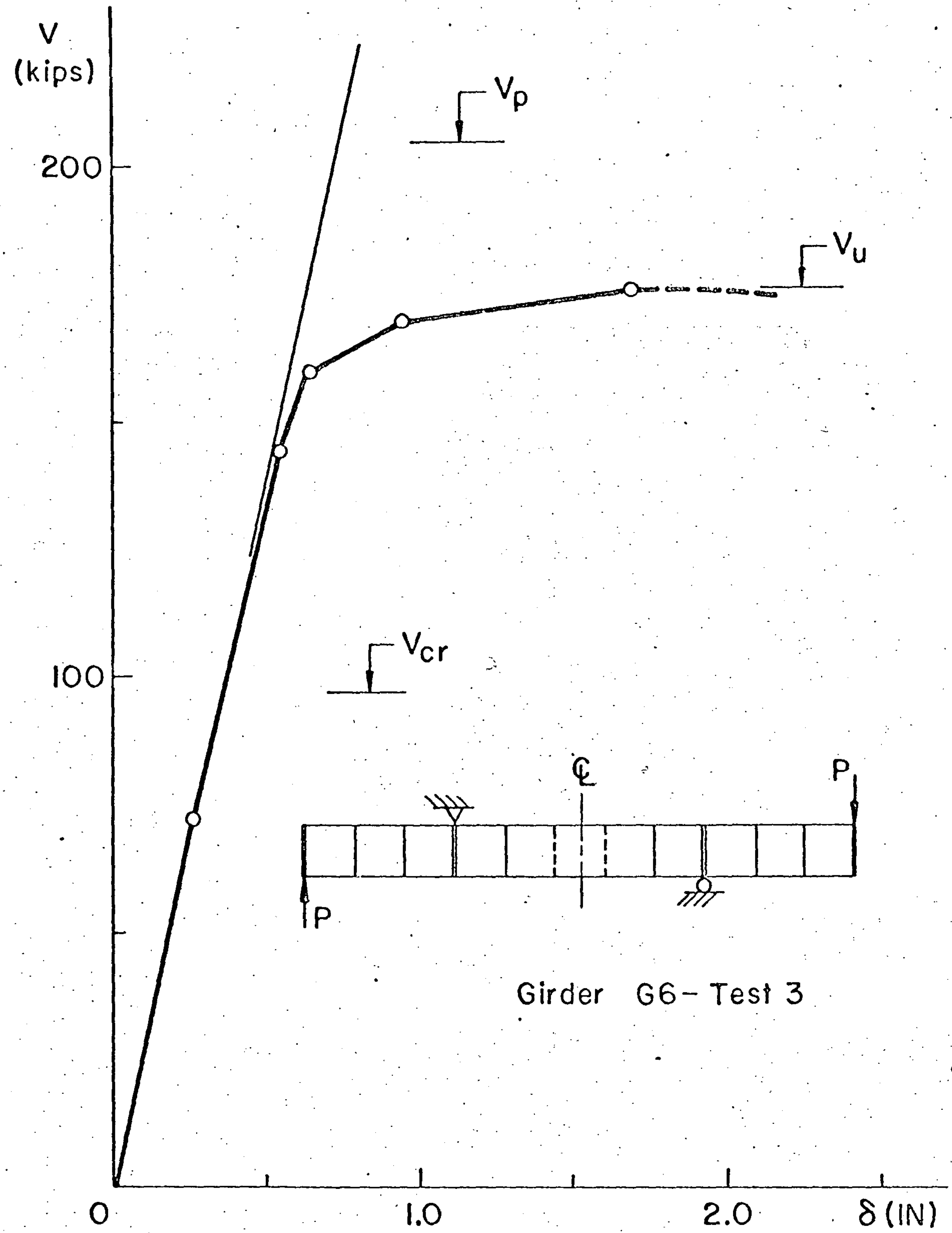


Fig. 17

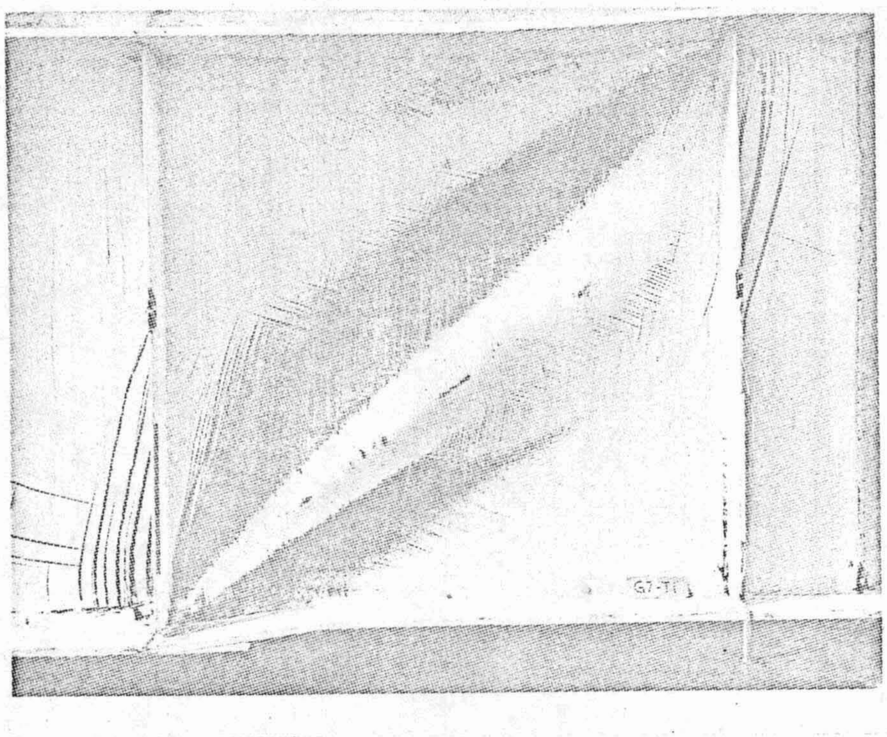


Fig 18

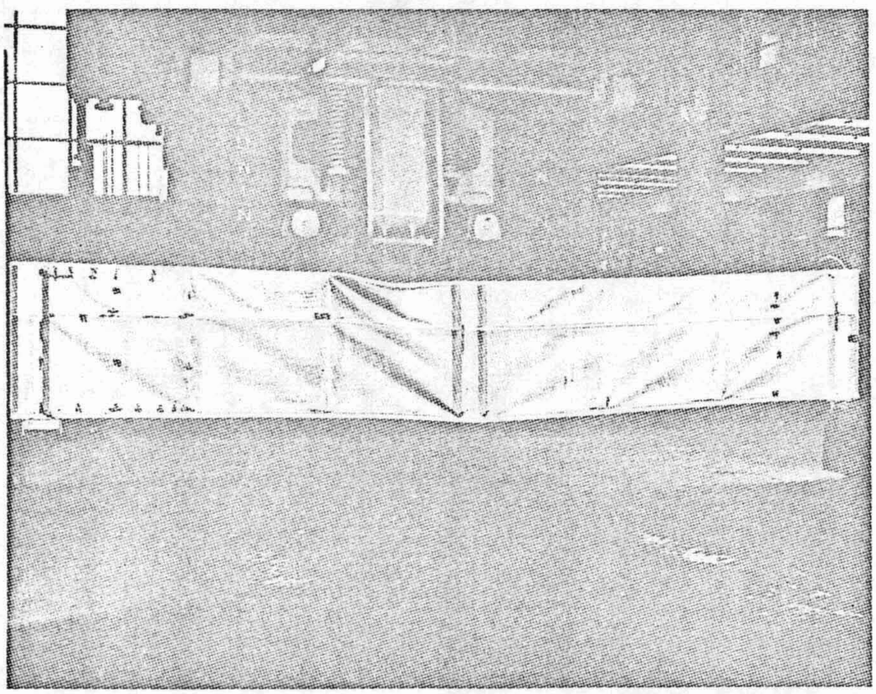
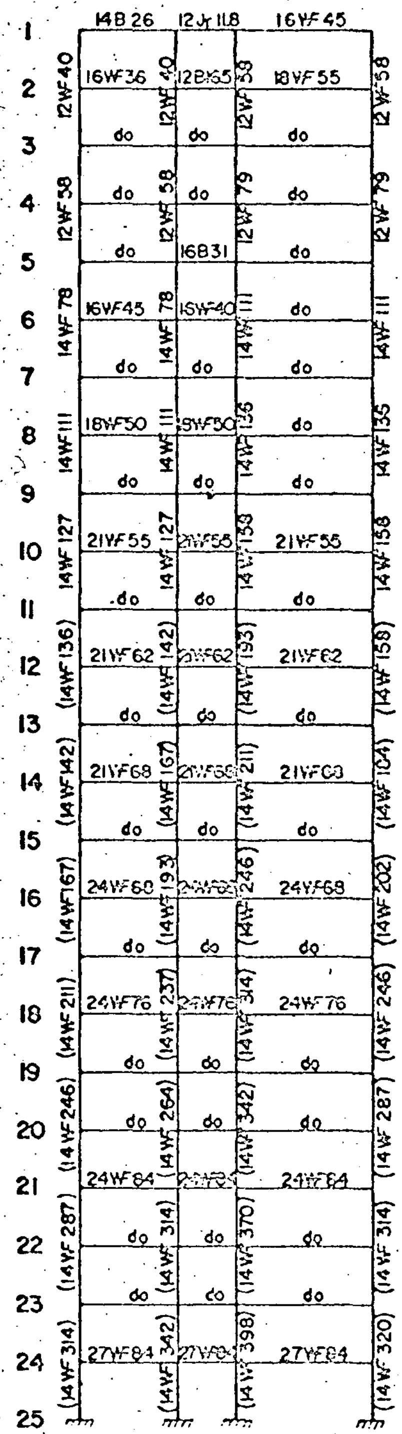


Fig 19

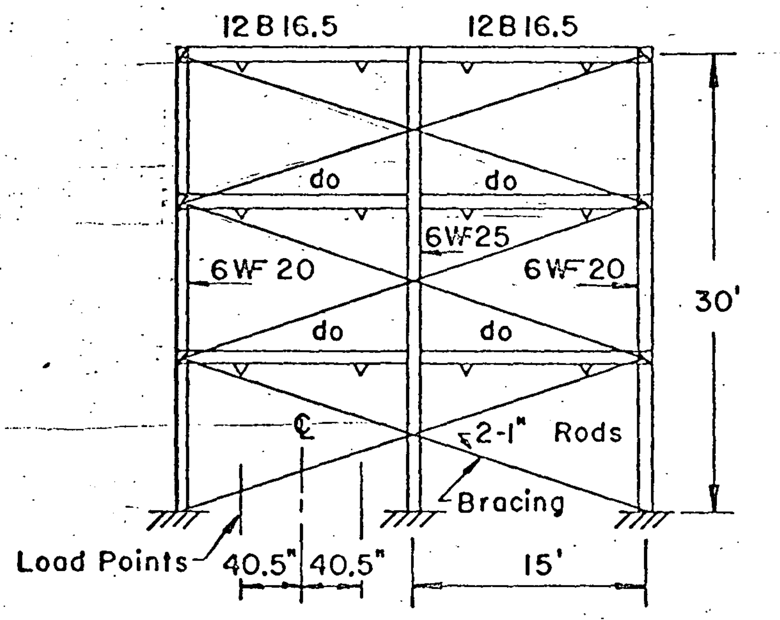


3⁰

Fig 20

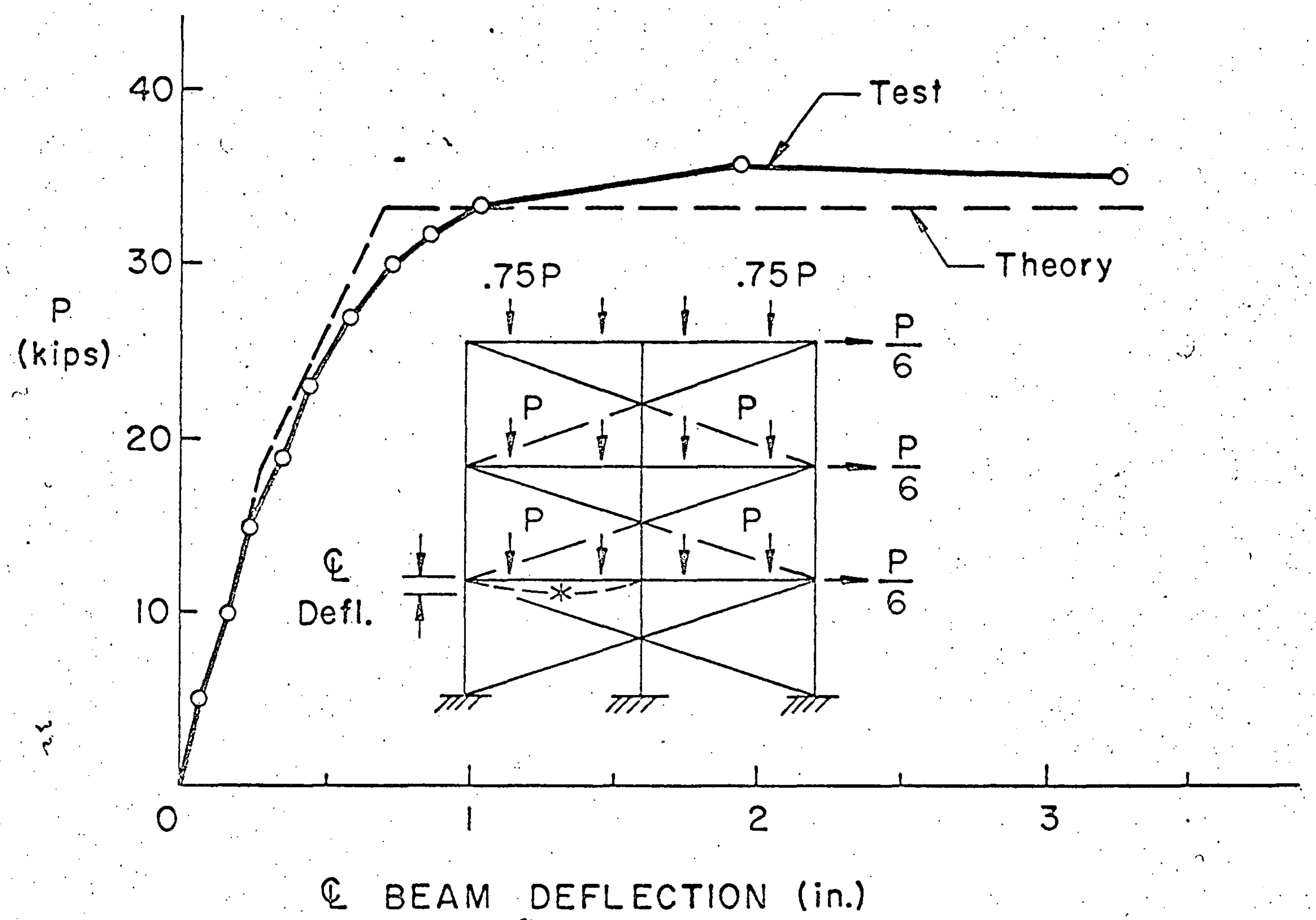
~~from p 19.25 of~~
~~273.20~~

~~273.100 M~~

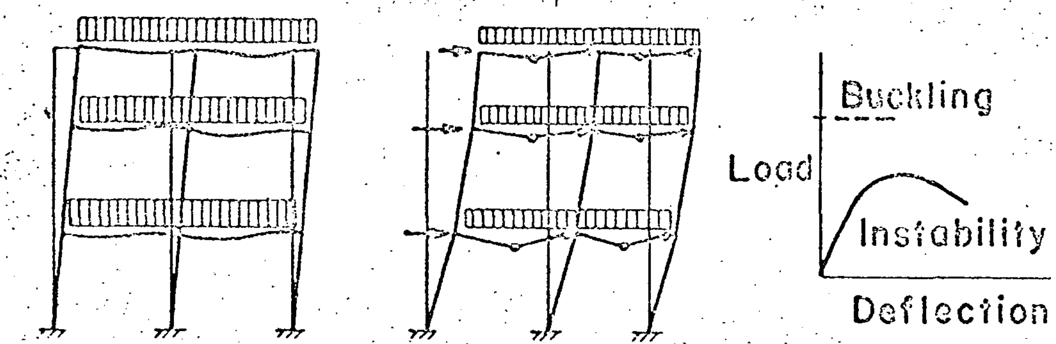

12 B 16.5
12 B 16.5
do
do
6WF25
6WF 20
6WF 20
do
do
30'
℄
2-1" Rods
Bracing
Load Points
40.5"
40.5"
15'

31

Fig 21


Test
Theory
40
30
20
10
P
(kips)
.75P
.75P
P
6
P
P
P
P
℄
Defl.
0
1
2
3
℄ BEAM DEFLECTION (in.)

- Fig 22 -



(a) Buckling (b) Instability (c) Load - Deflection

33

Fig. 23

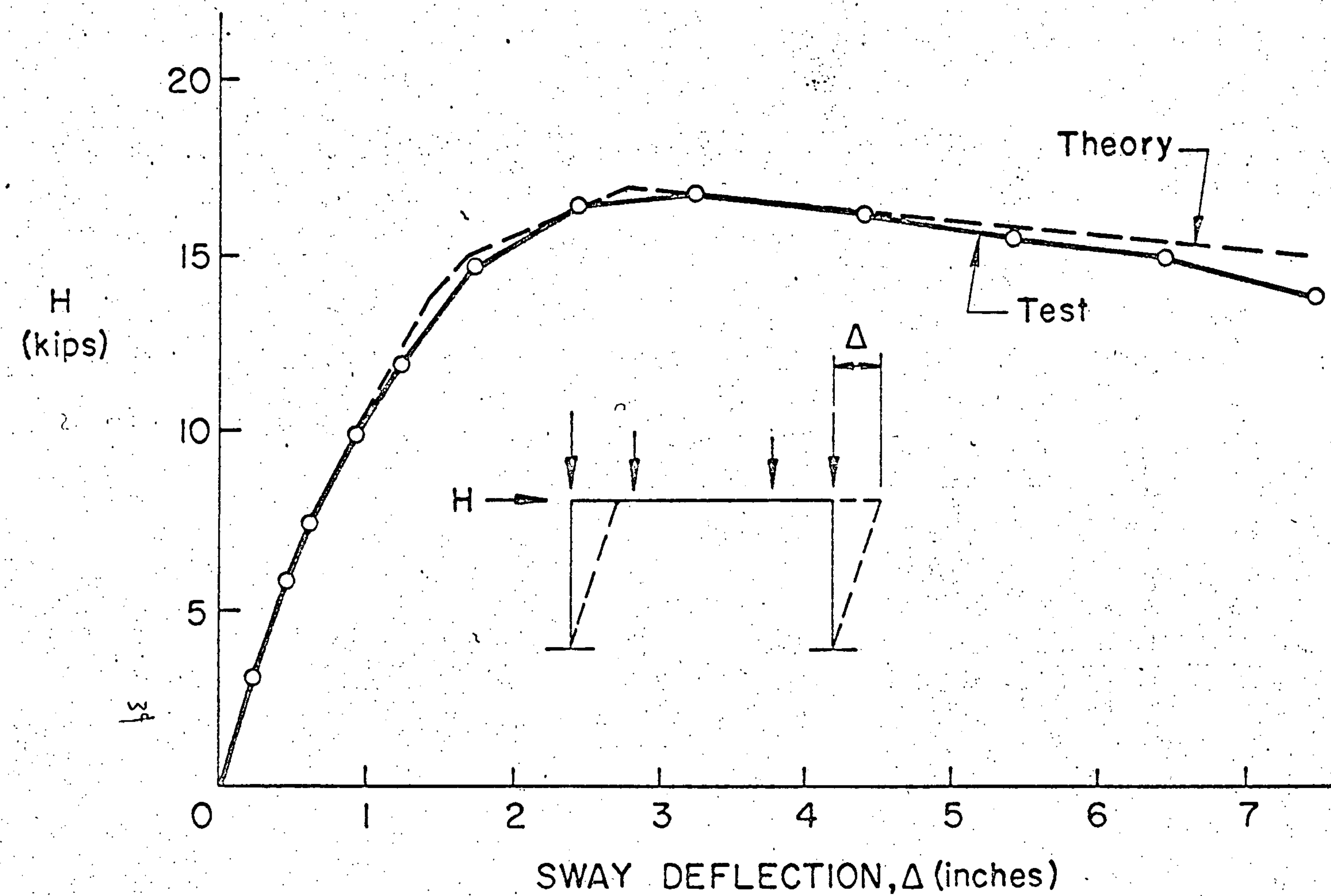


Fig 24

273.301

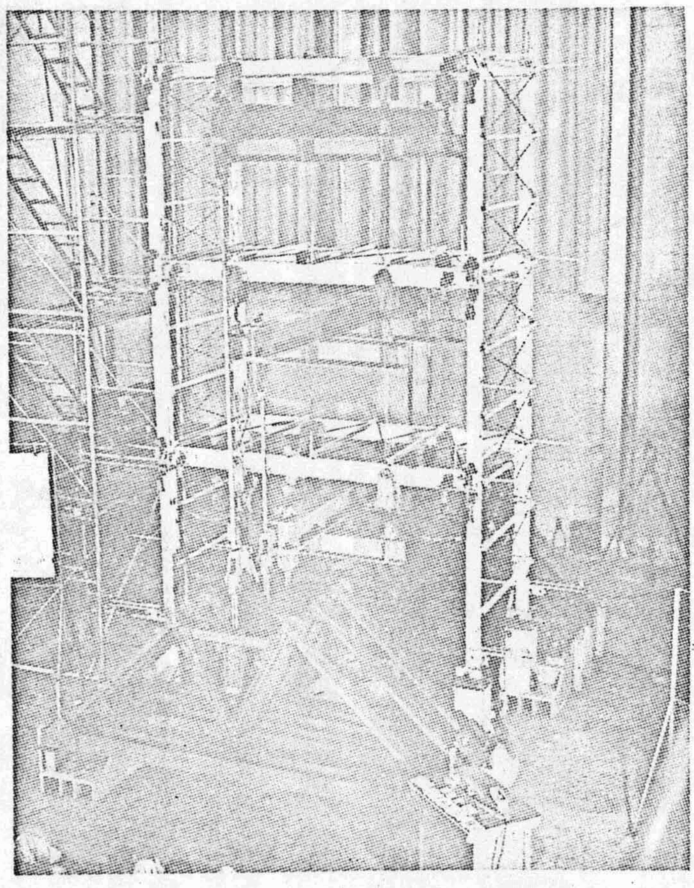
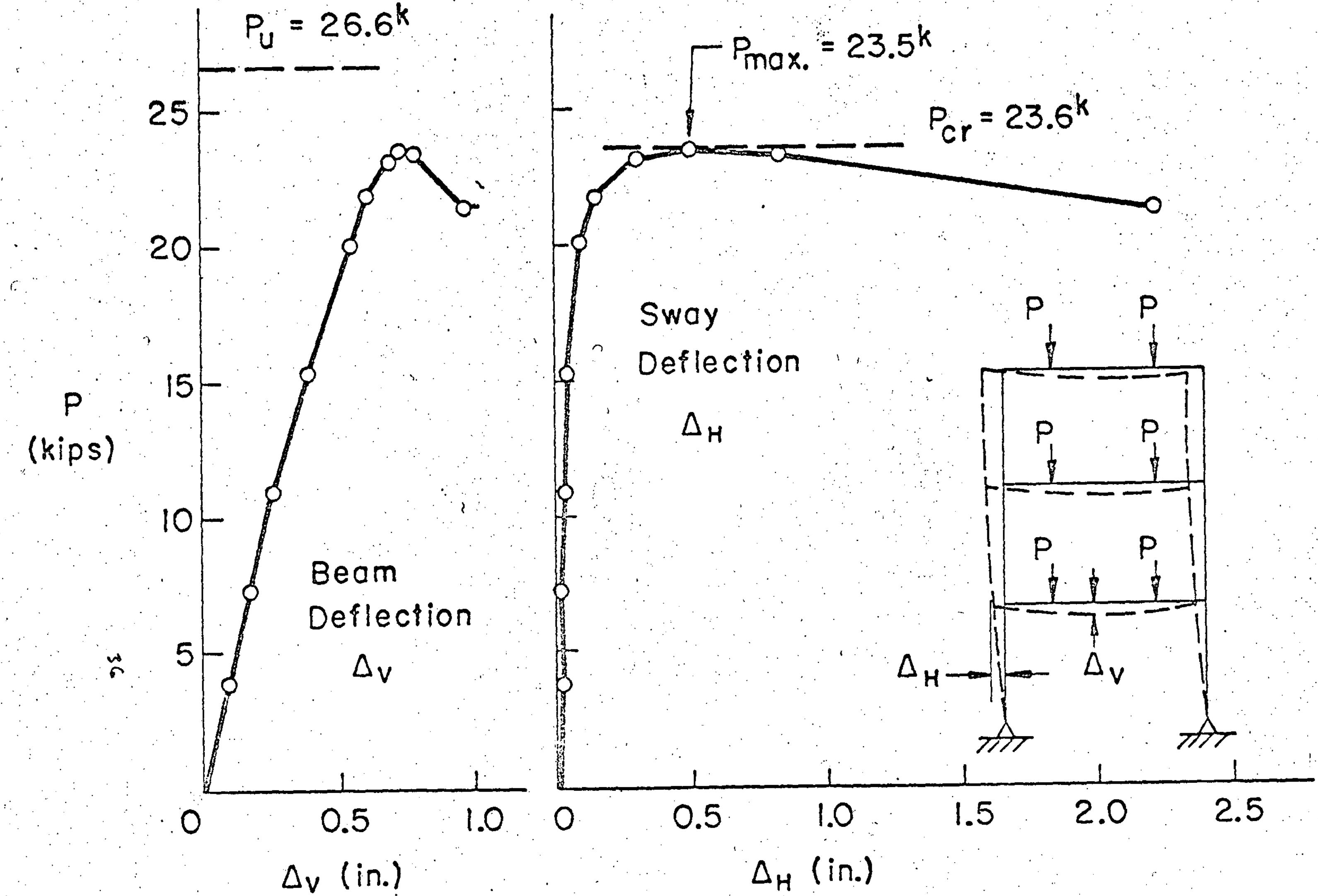


Fig 25



$P_u = 26.6^k$
$P_{max.} = 23.5^k$
$P_{cr} = 23.6^k$
P (kips)
25
20
15
10
5
Beam Deflection Δ_V
Sway Deflection Δ_H
P
Δ_H
Δ_V
0
0.5
1.0
1.5
2.0
2.5
Δ_V (in.)
Δ_H (in.)

fig 26.

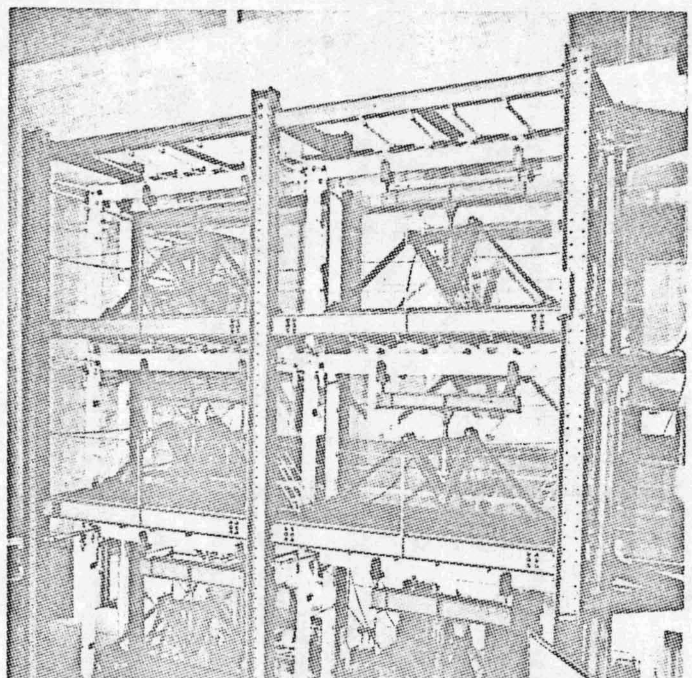


Fig. 27.

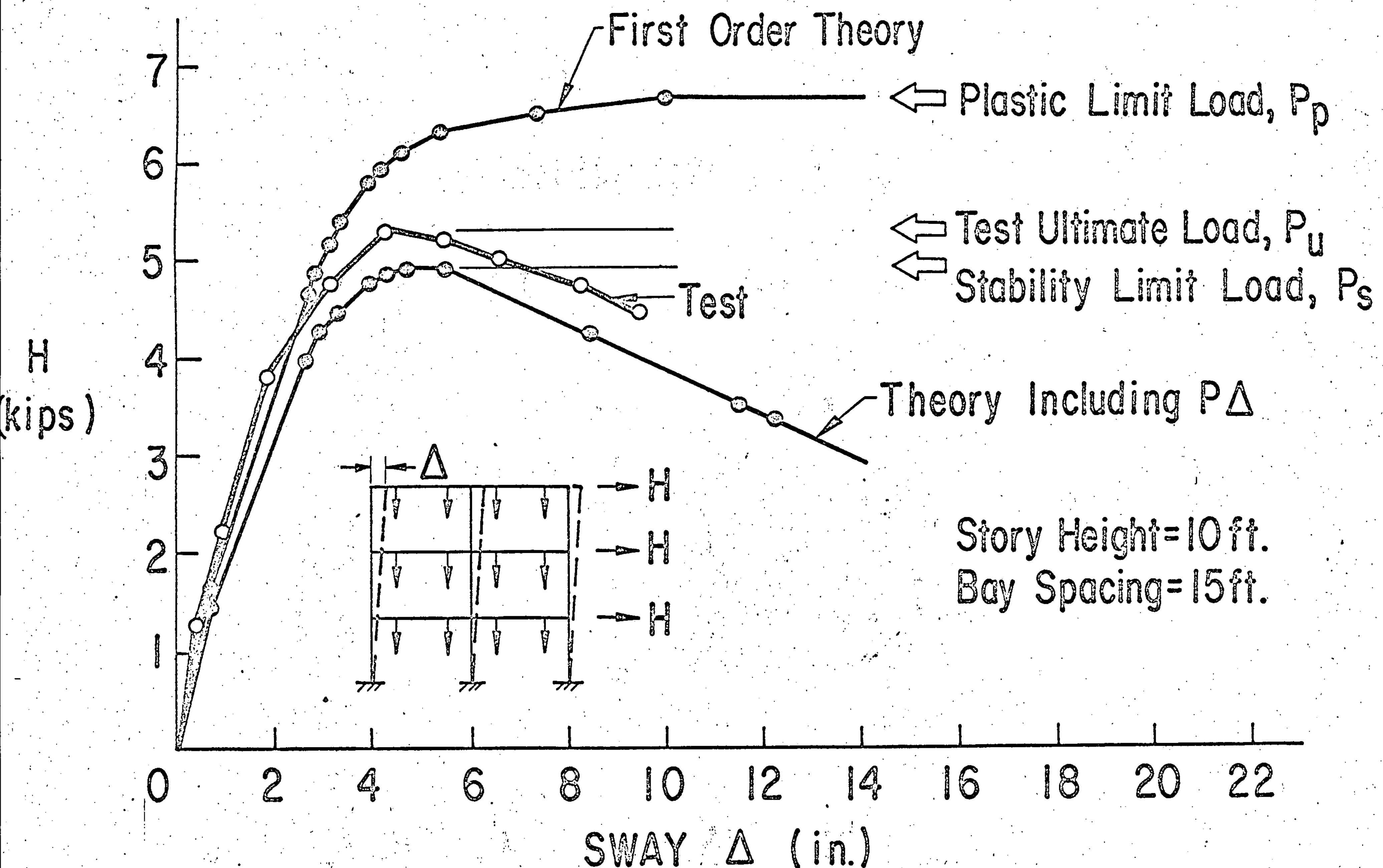
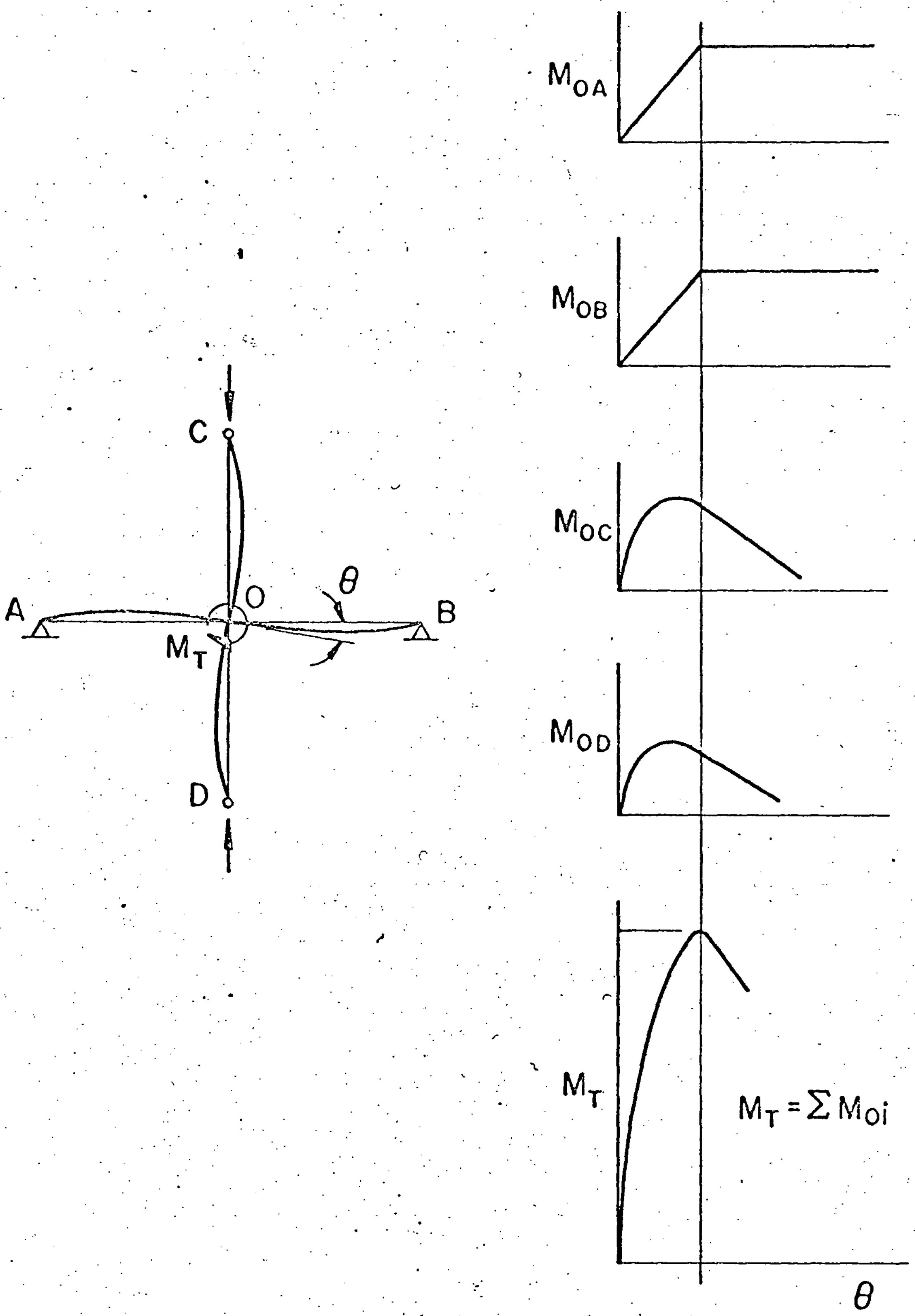
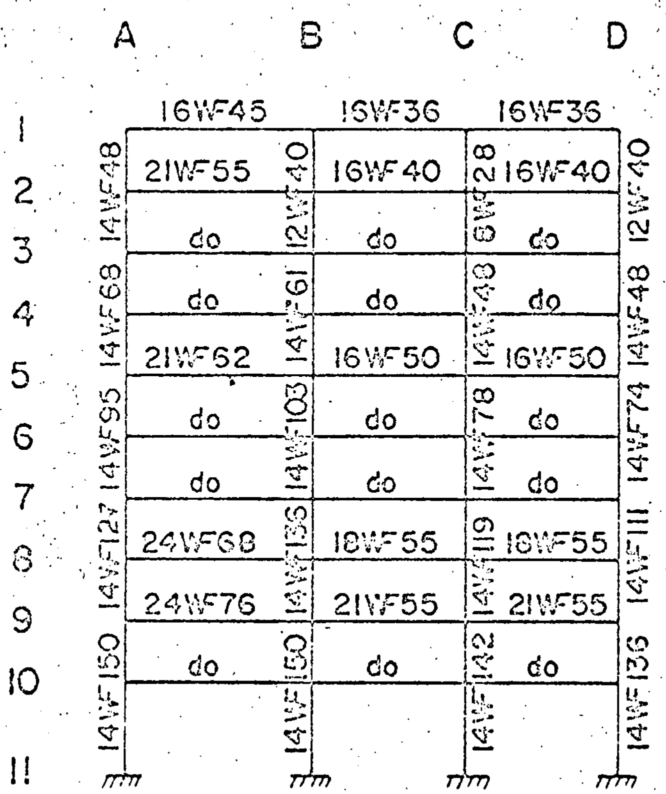

First Order Theory
Plastic Limit Load, P_p
Test Ultimate Load, P_u
Stability Limit Load, P_s
Test
Theory Including $P\Delta$
H (kips)
SWAY Δ (in.)
0
2
4
6
8
10
12
14
16
18
20
22
1
2
3
4
5
6
7
Δ
H
H
H
Story Height=10 ft.
Bay Spacing=15 ft.

Fig 28



35

Fig 29



A 36 Steel

Beam Wt. = 20.4^T

Column Wt. = 22.1^T

Total Wt. = 42.5^T

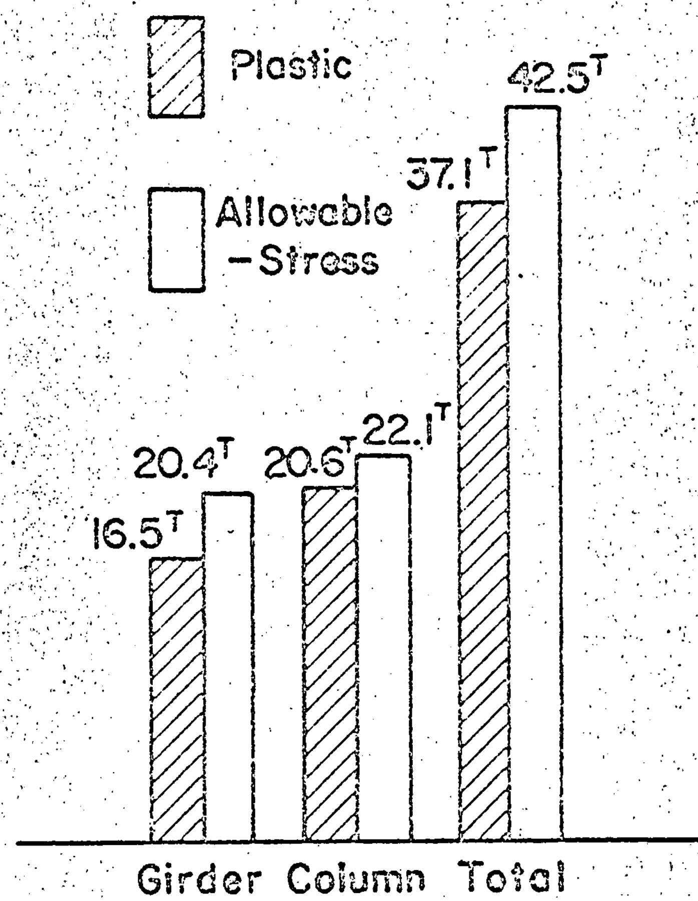


Fig 30

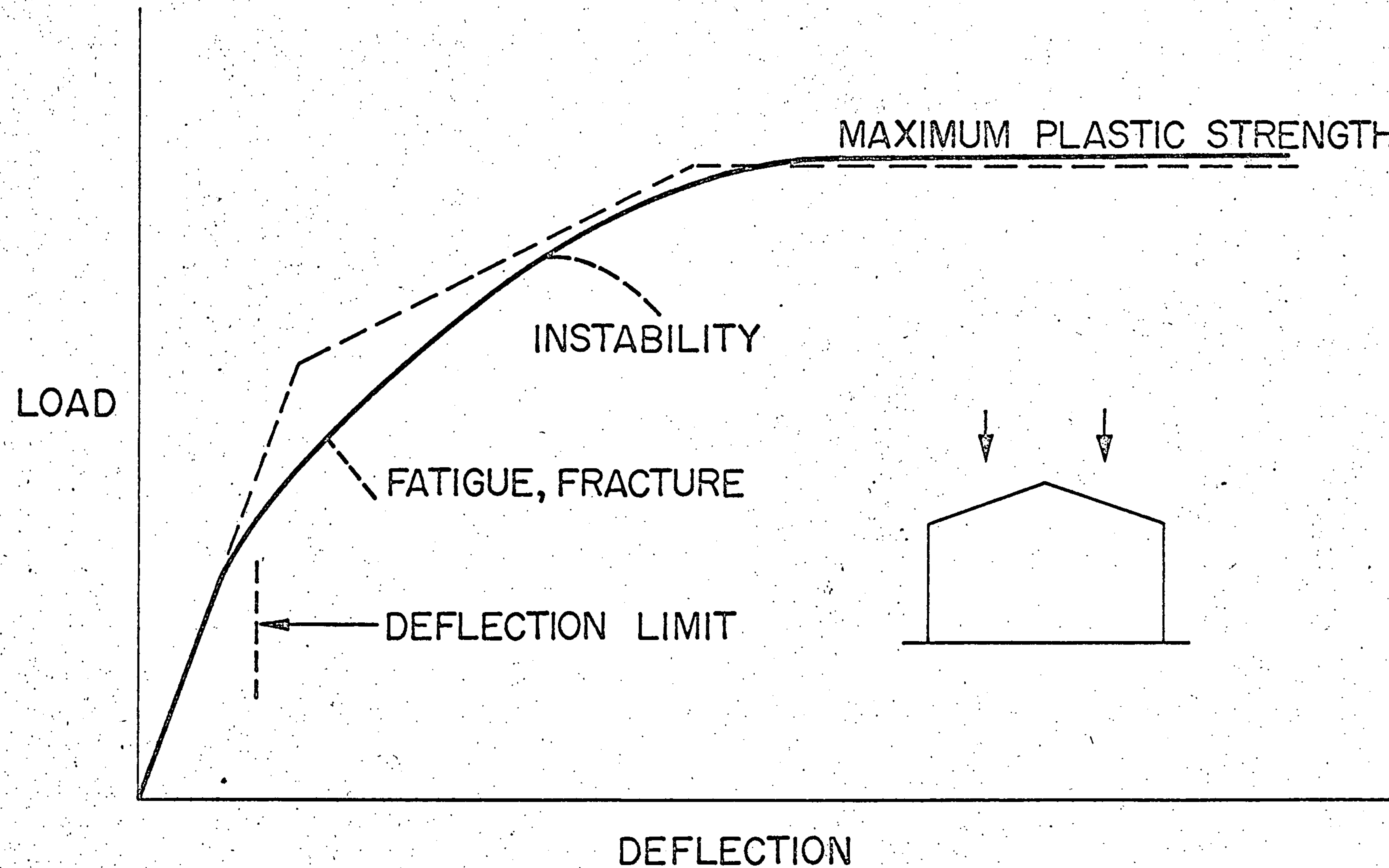

MAXIMUM PLASTIC STRENGTH
INSTABILITY
LOAD
FATIGUE, FRACTURE
DEFLECTION LIMIT
DEFLECTION

Fig 31

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~~† This article does not purport to reference the very significant work done outside the United States. In view of the nature of the conference, other papers in the reviews should be consulted for the most recent pertinent articles.~~

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